Master of Engineering in Foundation Engineering
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University of Moratuwa

A STUDY ON MOST SUITABLE SLURRY CUTOFF WALL MATERIAL TO MITIGATE SEEPAGE

IN VENDRASAN DAM, TRINCOMALEE

by

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A Thesis

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1 INTRODUCTION

1.1 Earthen dams and Failures

Earthen dams are constructed to store water for the purposes of human consumption, food production, electricity production, industrial use and flood protection. Failure of Earthen dams may be due to hydraulic failure, structural failure or seepage failure. Hydraulic failure may be due to overtopping, erosion of the upstream/downstream surface/toe and piping. Structural failures can occur in either the embankment or the appurtenances. Failure of a spillway, lake drain, or other appurtenance may lead to failure of the embankment. Cracking, settlement, and slides are the more common signs of structural failure of embankments.

Seepage Failure occurs due to the uncontrolled seepage in both velocity and quantity. Water permeating slowly through the dam and progressively erode the soil in the embankment or the foundation toward the reservoir. Eventually with increased seepage flow rate the direct connection is made to the reservoir causing the piping. Piping may occur through the dam or the foundation causing dam failure.

Seepage problems in large dams should be addressed in proper way on time to prevent before it becomes a massive disaster. Field investigation and data evaluation reveal the type of seepage and its extent. Then the most appropriate remedial measures can be adopted. Construction of toe filter, toe drain, downstream seepage berm, conducting cement/clay grouting and Cutoff walls construction are more frequently practiced to prevent seepage through and beneath the dam.

Cutoff walls make the seepage paths longer, decrease the exit gradient at the toe and reduce the seepage quantities. Compacted impervious trench cutoffs, concrete cutoff walls, sheet piles, slurry trenches/cutoff walls are some different types of cutoff walls currently being utilized.

1.2 Slurry Cutoff Wall

The slurry trench/cutoff method is well known for creating impermeable groundwater barriers and has been used for decades to create economical and positive cutoff walls in the core or foundation soils beneath dams and dikes of many types and sizes.

Slurry cutoff walls are non-structural walls construct underground to act as barriers to the lateral flow of water and other fluids. Slurry wall construction starts with the “slurry excavation technique”, which was developed in Europe and has been used in the United States since the 1940s. Principal applications of slurry walls other than seepage barriers in the foundations of water retaining structures are site dewatering and pollution control. Soil-Bentonite (SB), Cement-Bentonite (CB), Soil-Cement-

Bentonite (SCB) are the currently practicing basic types of slurry mixes in the industry.

Soil Cement Bentonite (SCB) Slurry walls are a variation of the more common soil bentonite (SB) slurry walls. In this method, the soils excavated from the trench are blended with bentonite and cement to provide additional strength to the final backfill.

A detailed literature review is presented in chapter 2.

1.3 Vendrasan Dam in Trincomalee

The Vendrasan dam, owned by the Irrigation Department, is located south-west of Trincomalee and a short distance from the Kantale tank in eastern Province of Sri Lanka. The tank, of ancient origin, controls only a small catchment area of 11 km but is fed by water issued from Kantale tank. The capacity of the reservoir at Full Supply Level (FSL) is 25.7 mcm^{3}.The primary function of the scheme is the provision of water for irrigation of a large plain which is under intensive cultivation.

The homogenous earth fill dam is about 700 m long and has a maximum height of 16m. A curved concrete wall (overflow section) with a length of approximately 35 m which serves as spillway is located in a wooded area at the southern, right-hand end of the Vendrasan dam. The sluice is situated close to the left abutment at approximately Ch 00+047.

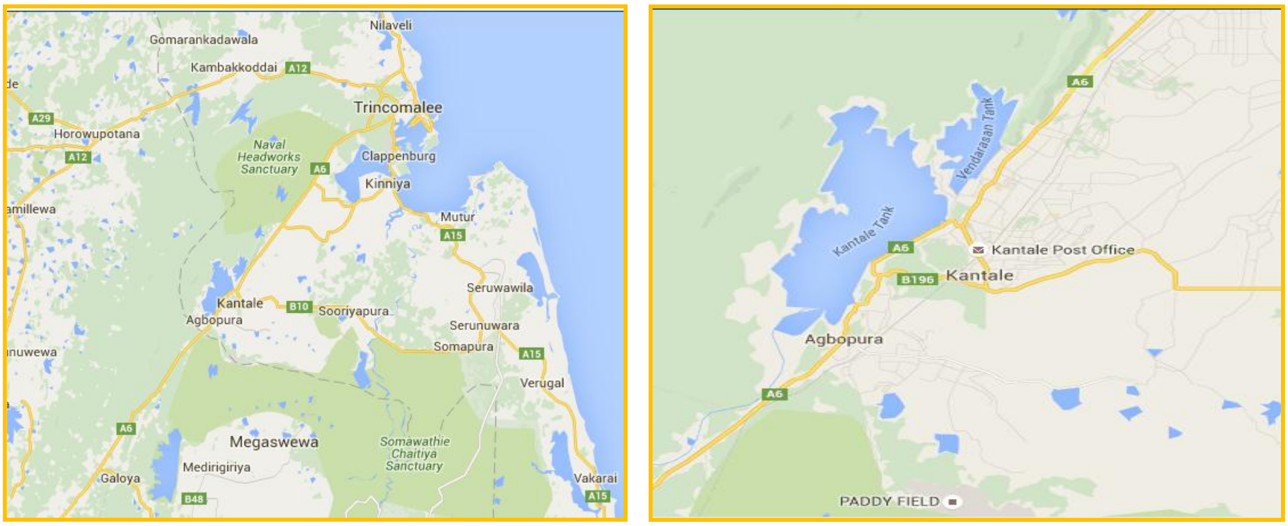


Figure 1.1 – Google images of Vendrasan Dam

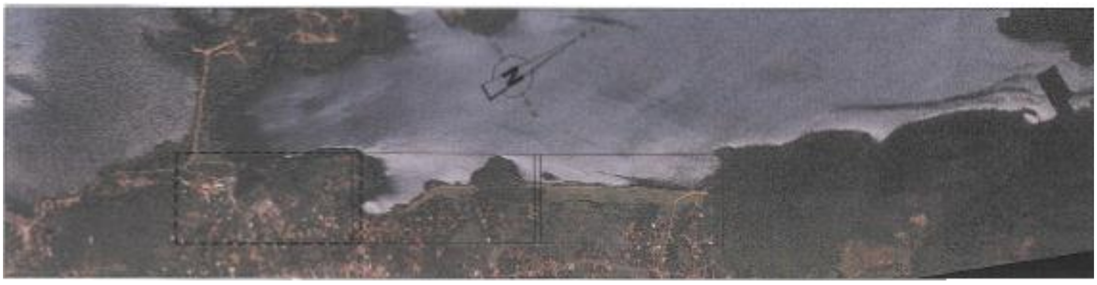


Figure 1.2 – Satellite image of Vendrasan Dam

The tank bund has been modified a number of times, and still suffers from toe seepage, to such an extent that the water level in the tank is currently held at a low level, several meters below FSL.

The main findings of the embankment inspection made on January 2013 can be summarized as below.

- Evidence of seepage areas along the dam toe. It is reported that there is excessive seepage and the tank water level is maintained below FSL, for safety reasons.
- Appearance of sinkholes in existing stabilizing berm on the downstream slope. Sinkholes indicate the lack of appropriate filter layers in existing drainage system.
- Inadequacy of embankment cross section (steep upstream slopes, insufficient width of crest) at certain locations.
- Displaced or missing rip-rap along most of the upstream slope of the left part of the dam.
- Localized deficiencies of crest and crest shoulders.
- Dense vegetation along the dam toe.

The critical seepages were observed between the chainages of 470 m – 590 m along the embankment and from the geological investigations, it was revealed that unacceptable GM, SM and SP materials are present in the heterogeneous filling which was done at past. It is clear that these permeable layers pave path to the seepages at toe.

Hence, it is required to cutoff or lowers the high level phreatic line created with more permeable layers to address this issue. For that, slurry mixtures consisting of Soil, Bentonite and Cement materials can be utilized in suitable proportions by achieving required strength and permeability.

Slurry wall techniques are well practiced by many countries successfully in decades, but still not in Sri Lankan engineering context. Once it is proved the effectiveness it may be useful for future planners and designers to incorporate this technique in rehabilitation works and also where applicable.

Therefore this study is focus on investigating the suitability of Soil Cement Bentonite (SCB) slurry cutoff wall material to mitigate seepage in Vendrasan dam Trincomalee.

1.4 Objectives

1) Carryout a comprehensive literature study on slurry cutoff wall techniques

2) Investigate the applicability of Soil Cement Bentonite (SCB) slurry cutoff wall to mitigate seepage in Vendrasan dam Trincomalee by utilizing SEEP /W Software

3) Determine a suitable mix design of Soil Cement Bentonite (SCB) cutoff wall backfill which shall fulfill the permeability and strength criteria to introduce to the Vendrasan dam Trincomalee

4) Evaluate the suitability and seepage after introducing the appropriate slurry cutoff wall.

2 LITERATURE REVIEW

Considerable researches have been conducted on Soil Bentonite (SB) slurry wall which is more often used to provide barriers to the lateral flow of groundwater. But only limited researches done on Soil-Cement-Bentonite (SCB) slurry walls were found. But these SCB walls are now used increasingly in recent years where the strength of a normal soil-bentonite wall would be inadequate to carry foundation loads. The addition of cement to the backfill blend allows the backfill to set and form a more rigid system that can support greater overlying loads. This literature review will follow the background information of Soil-Bentonite (SB) cutoff walls, Soil-Cement-Bentonite (SCB) cutoff walls and also the successfully conducted SCB slurry cutoff walls in recent past.

It is generally believed that the concept of excavating under bentonitic supporting slurry was first developed by Veder, in Austria, in 1938. According to Xanthakos (1979), the first slurry trench cut-off was "probably" built at Terminal Island, near Long Beach, California in 1948. It was 45 feet deep and backfilled with soil. Ryan and Day (2003) reported that "thousands" of such walls have been built in the U.S. since the early 1970s, predominately backfilled with soil-bentonite.

Soil-Cement-Bentonite (SCB) slurry walls are an adaptation of traditional soil- or cement-based walls. Fundamentally, the SCB wall is a soil-bentonite slurry wall with cement added to the backfill (less than 10%). The benefit of the SCB slurry wall is that it is similar to the cement bentonite wall in strength and to the soil-bentonite wall in hydraulic conductivity (Rumer et al., 1996).

Soil-Cement-Bentonite slurry walls (SCB wall) are constructed in much the same manner as a conventional Soil Bentonite walls (SB wall) (Ryan CR, 1984).

Designing the SCB backfill is a complex issue involving conflicting actions of the various materials involved. While the SCB wall provides additional strength, permeability is one property that generally suffers in comparison to soil-bentonite slurry walls. A normal permeability specification would be a maximum of 1×10^{-6} cm/sec. With special attention to materials and procedures, a specification of a maximum 5×10^{-7} can be achieved (Ryan & Day, 2003).

2.1 Soil-Bentonite (SB) Cutoff Walls

2.1.1 Construction and Design Procedure

Soil-Bentonite slurry cutoff walls are the more common and frequently used cutoff wall technique in the past. Thousands of such walls have been constructed for number of purposes. These walls are constructed using the slurry trench method.

Typically 3 to 5 feet wide narrow trench is excavated under the bentonite-water slurry that is used to support the trench walls from collapsing. The bentonite-water slurry is typically 4% to 6% bentonite by weight (*Barrier* 1995). The slurry is kept at an elevation higher than the water table in the adjacent soil. This causes the slurry to flow into the adjacent soil, forming a thin layer of bentonite at the trench wall, which is referred to as a "filter cake" (Filz et al. 1997). The lateral pressure from the slurry in the trench acts against the filter cake and provides a stabilizing force.

Excavation of the trench is typically performed with a backhoe with a modified boom to depths of 60 feet, and with clamshells for deeper depths (*Barrier* 1995). As excavation proceeds along one end of the trench, the trench is backfilled with soil-bentonite at the other end. Initially, the soil-bentonite must be placed at the bottom of the trench with a clamshell until the backfill reaches the ground surface and creates a ramp as shown in Figure 2.1 in page 13. Subsequently, soil-bentonite can be pushed into the trench and be allowed to slide down the slope of the existing backfill. The soil-bentonite displaces the bentonite-water slurry, since it has a higher density, and becomes the final cutoff wall backfill.

The final backfill soil-bentonite is a mixture of the soils excavated from the trench or off-site soil and bentonite-water slurry. The soil-bentonite is typically mixed next to the trench with a bulldozer and a mixing pit or a pugmill may also be used. The soil-bentonite generally has a hydraulic conductivity of $1x10^{-7}$ to $1x10^{-8}$ cm/s (*Barrier* 1995).

Design of a soil-bentonite cutoff wall first involves establishing the alignment and the depth of the wall. This is determined based on the purpose of the cutoff wall and the site specific geology and hydrology. Soil-bentonite cutoff walls are typically keyed into an impervious layer to prevent seepage under the wall. If an upward gradient exists or can be created, or if contaminants are less dense than water, it may not be necessary to key the wall into an impervious layer; these types of cutoffs are referred to as hanging walls.

The thickness of the wall is typically 2 to 5 ft, which corresponds to typical widths of a backhoe bucket (D'Appolonia 1980). Evans (1995) recommends that if walls will be exposed to high hydraulic head conditions, such as beneath a dam, they should be analyzed for hydraulic fracture. If hydraulic fracture is a concern, a thicker wall is recommended. Although detailed design procedures are not available for analysis of hydraulic fracture of soil-bentonite cutoff walls, some rule-of-thumb approaches do exist, such as the U.S. Army Corps of Engineers' recommendation that soil-bentonite cutoff walls be at least 0.1 ft wide for every foot of head difference (U.S. Army Corps of Engineers' 1986).

In developing specifications for construction of soil-bentonite cutoff walls, emphasis is placed on proper construction quality control. The following items are typically specified: contractor qualifications, bentonite material properties, water properties, bentonite-water slurry properties, soil-bentonite properties, trench excavation procedures, and soil-bentonite backfill mixing and placement procedures (U.S. Army Corps of Engineers' 1996). Properties of soil-bentonite are typically specified in order to achieve a low hydraulic conductivity cutoff wall. Properties of bentonite-water slurry are typically specified in order to maintain a stable trench during excavation. Recommended ranges of property values can be found in many references (D'Appolonia 1980; Evans 1995). The recommended values are mostly based on past experience.

Review of the literature indicates that current construction and design procedures are based on experience in order to achieve a soil-bentonite cutoff wall that is easily constructible, stable, and exhibits a low hydraulic conductivity.

2.1.2 Engineering properties of soil-bentonite

Soil-Bentonite mixtures are very difficult to characterize because it can vary greatly. One reason for the variation is that soil-bentonite is typically made by mixing soil excavated from the trench with bentonite-water slurry, and the excavated soil can vary greatly from site to site or even across a particular site.

Engineering properties concerned in designing soil-bentonite found in the literature basically focus on hydraulic conductivity, compressive strength, compressibility and deformation characteristics. The primary goal is to provide a cost-effective, low permeability material. In addition, a relatively low compressibility soil-bentonite mixture is desirable in order to prevent excessive settlement in the trench and reduce adjacent ground deformations.

There are several recommendations on grain size distributions of the soil-bentonite in order to achieve these goals. D'Appolonia (1980), states that a soil-bentonite will have low compressibility if there are enough granular particles to have grain to grain contact. For both low compressibility and low permeability, a well graded material with gravel through clay sized particles is recommended (D'Appolonia 1980; Evans 1995). D'Appolonia (1980) recommends a granular matrix with 20% to 40% plastic fines and a minimum of 1% bentonite. Evans (1995) recommends a well graded matrix with sand and gravel, 20% to 50% fines, and 1% bentonite. They also state that other gradations such as fine sands and clays have also been used successfully.

For best placement consistency, the recommended slump is 4-6 inches (Evans 1991, Millet et al. 1992) or 2-6 inches (D'Appolonia 1980). The slump is measured with a standard concrete slump cone apparatus.

Compressibility Properties of Soil-Bentonite

D'Appolonia (1980) plots the compression ratio versus fines content for various soil-bentonite mixtures as shown in Figure 2.2. The compression ratio is defined as;

$$\text{Compression Ratio} = \frac{C_c}{1+e_o}$$

where: C_c = Compression index = $\dfrac{\Delta e}{\Delta \log \sigma'}$

e_o = initial void ratio

Δe = variation of void ratio

$\Delta \log \sigma'$ = variation of effective stress

The compression ratio corresponds to the stress range from 0.5 to 2 kg/m^2. Data from both one dimensional compression and isotropic compression is included in the Figure. It can be seen that the compressibility increases with fines content. Also, soil-bentonites with plastic fines are more compressible than soil-bentonites with non-plastic fines. In general, a soil-bentonite with 20% to 40% fines has a compression ratio between 0.02 and 0.07 for the stated stress range. It can also be seen that soil-bentonite in one-dimensional compression has a higher compression ratio than in isotropic compression.

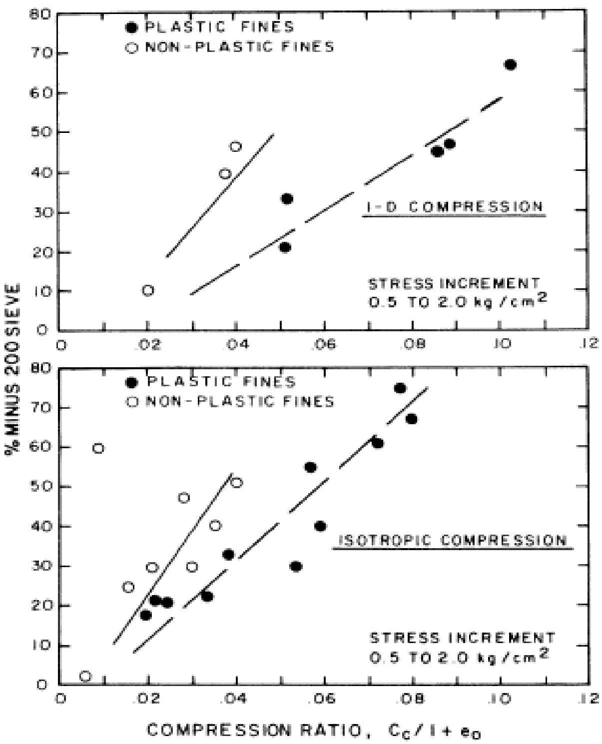


Figure 2.2 Compression ratio Vs fine content for varios soil bentonite mixtures
Source: D' Appolonia (1980)

Khoury et al. (1992) present data from a soil-bentonite cutoff wall constructed in an earth dam. Several different soil-bentonite mixtures were tested. The mixtures were prepared with various grain size distributions to represent the range of onsite backfill material. The information is summarized in Table 2.1. The compressibility increases with fines content, following the same trend as D'Appolonia's data (1980). The void ratios or stress increment associated with the compression and swell ratios were not given. It appears that the mixtures tested by Khoury et al. (1992) are slightly more compressible than those reported by D'Appolonia (1980).

Table 2.1 – Consolidation data on various soil-bentonite mixtures
Source: Khoury et al. (1992)

Soil-Bentonite Property	Mix 3	Mix 4	Mix 5
Percent Bentonite	1.05	1.18	1.65
Percent Passing No. 200 Sieve	30.5	37.6	72.5
Water Content (%)	31.0	39.0	62.1
Compression Ratio	0.077	0.091	0.137
Swell Ratio	0.005	0.006	0.010
Coefficient of Consolidation for 2000-4000psf (ft^2/yr)	292	215	70

Evans and Cooley (1995) present consolidation data from undisturbed samples taken from a 4 year old and a 10 year old soil-bentonite wall. The consolidation information is presented in Table 2.2. The compression ratios are more similar to those reported by Khoury et al. (1992) than to those reported by D'Appolonia (1980).

Table 2.2 – Consolidation data from undisturbed samples
Source: Evans and Cooley (1995)

Age of Wall (Years)	Sample Depth (Feet)	Compression Ratio	Swell Ratio
4	7	0.088	0.006
4	9	0.108	0.009
4	14.5	0.147	0.006
10	9.5	0.110	0.015
10	13	0.097	0.007

Permeability of Soil-Bentonite (SB)

The permeability of a SB/SCB cut off wall is a function of both the filter cake that forms on the trench wall and the permeability of the backfill placed in the trench. The relative contribution of each constituent depends on the relative permeability and thickness of the two materials.

D' Appolonia and R. Ryan in 1979 derived following formula for the horizontal permeability of a cut-off wall from Darcy's low and a continuity equation:

$$Q = k\,i = k\frac{\Delta h}{(2t_c + t_b)} = k_c\frac{\Delta h_c}{2t_c} = k_b\frac{\Delta h_b}{t_b}$$

and $\Delta h = \Delta h_c + \Delta h_b$

Where; Q = flow rate

k = permeability

Δh = head loss

k_c / t_c = Permeability /thickness of filter cake

k_b / t_b = Permeability /thickness of backfill

Combining equations and considering that $t_b >> t_c$ leads to:

$$k = \frac{t_b}{\left(\frac{t_b}{k_b}\ \frac{t_c}{k_c}\right)}$$

The permeability of the backfill material can be determined in a laboratory test. The thickness of the backfill is selected in design. The ratio k_c/t_c can also be determined experimentally under simulated field condition. For a wide variety of practical applications, the ratio k_c/t_c varies between the relatively narrow limits of 5 to 25 * 10^{-9} cm/sec (D' Appolonia and R. Ryan in 1979).

The overall cutoff permeability and backfill permeability for typical values of k_c/t_c is theoretically plotted as in Figure 2.3. According to the plot the effect of the filter cake permeability on the overall average permeability is less if the backfill permeability is low.

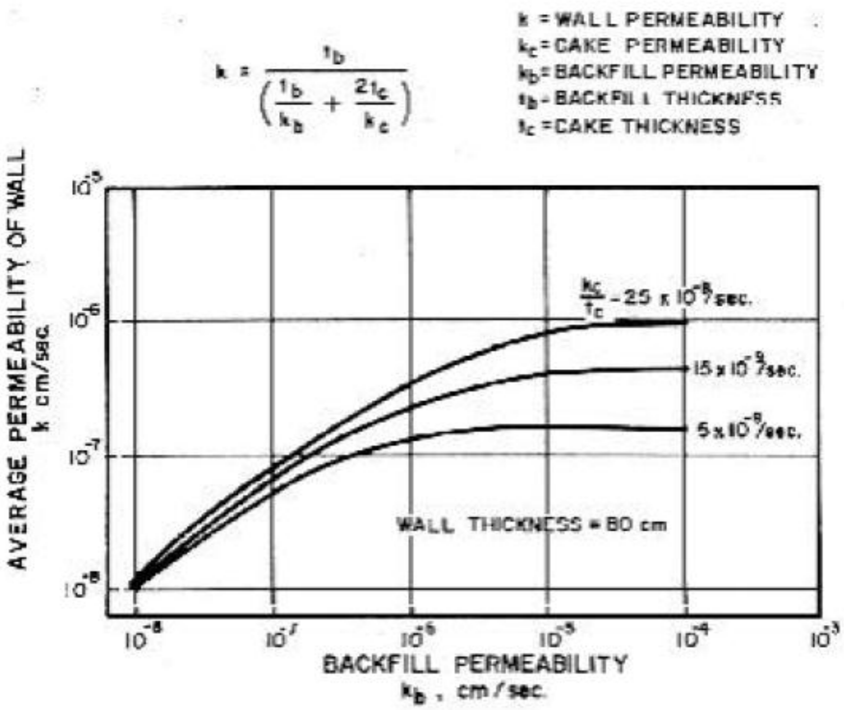


Figure 2.3: Theoretical relationship between wall permeability and permeability of the filter cake and backfill

Source: D' Appolonia and R. Ryan (1980)

The quantities of bentonite added and fine content of soil (passing No.200 sieve) control the permeability of soil bentonite backfill. It is illustrated with field data in Figure 2.4 and Figure 2.5 respectively.

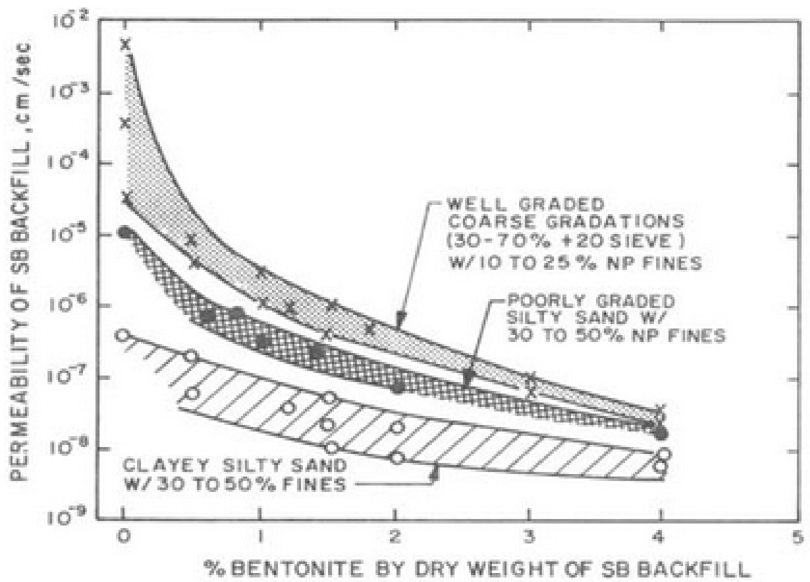


Figure 2.4 : Relationship between permeability and quantity of bentonite added to SB backfill

Source: D' Appolonia and R. Ryan (1980)

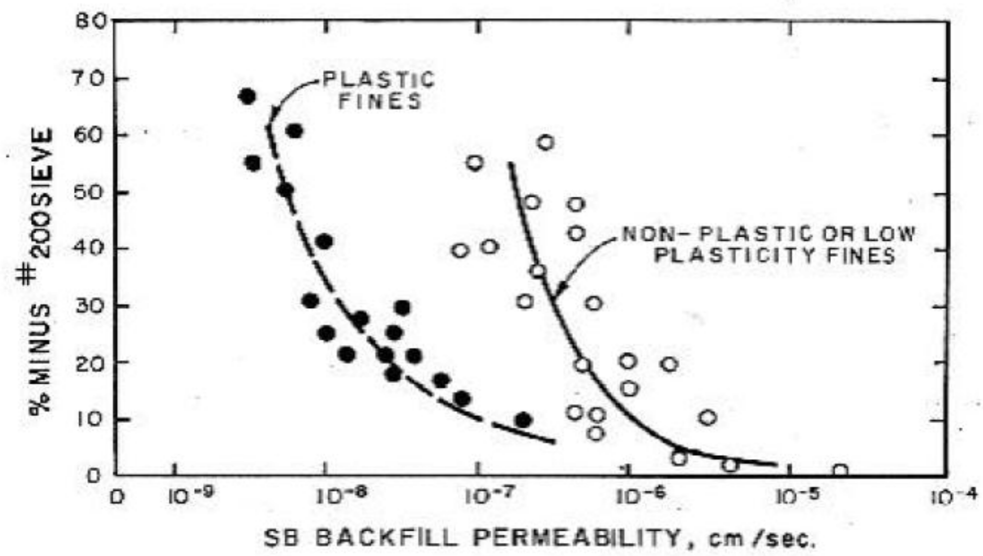


Figure 2.5 : Permeability of Soil - Bentonite backfill related to fines content
Source: D' Appolonia and R. Ryan in 1980

It is well established that testing conditions should simulate field conditions and that field stress is an important consideration. It is also well established that an increase in confining pressure will cause a reduction in hydraulic conductivity, and that the effect is more pronounced with more compressible soils, such as soil-bentonite than less compressible soil, such as compacted sand-bentonite liners (Evans 1995). If values of confining stress that are used in testing soil-bentonite are greater than those in the field, the hydraulic conductivity may be significantly underestimated for a cutoff wall. The selection and use of an appropriate confining pressure for hydraulic conductivity tests on soil-bentonite remains an unresolved issue.

The most important property of Soil-Bentonite backfill is the low permeability. Typically Soil-Bentonite backfill has a permeability in the range of 10^{-6} to 10^{-8} cm/sec. Environmental projects often require a permeability less than 1 x 10^{-7} cm/sec, but a levee or dewatering project may require a permeability less than 1 x 10^{-6} cm/sec. Either value is achievable with the right mix of materials.

Strength Properties of Soil-Bentonite

Soil-Bentonite backfill has low strength and will remain soft (in the range of 300 psf (15 kPa)) for the design life, but this is nearly always sufficient to maintain a vertical cut through the wall for subsequent installation of utilities and other light structures. Larger surface loadings like roads and structural foundations require the removal and replacement of the top few feet of the wall. Sometimes geogrids are used to distribute the loads, above the wall to the adjacent soil of the wall.

Researchers at Bucknell University conducted a suite of in situ tests on a Soil-Bentonite cutoff wall in 2008. Cutoff wall properties were measured in situ employing cone penetration tests (CPT), Marchetti dilatometer tests (DMT), vane shear tests (VST), and ground water level monitoring on both sides of the wall. Tests were conducted during construction and at times of 3 months, 6 months and 9 months after construction to evaluate the change in wall properties with time. The VST and CPT showed an increase in backfill shear strength over the time-frame of the study. A slight increase of shear strength with depth was also found.

All of the reported values of effective friction angle for soil-bentonite mixtures are between 31 and 33 degrees (Evans JC at al. 2010).

2.1.3 In Situ State of Stress in Soil-Bentonite Backfill

The stress state of soil-bentonite significantly influences the measured hydraulic conductivity. Many authors agree on the need for greater understanding of the state of stress in soil-bentonite cutoffs (Khoury et al. 1992; Evans 1995; Filz 1995).

When the soil-bentonite is initially placed into the trench, the water content is very high and the strength of the soil-bentonite is very low; it flows into the trench. As the trench is filled from the bottom up with soil-bentonite, it takes time for the soil-bentonite to consolidate and feel the effective stresses produced by the soil-bentonite above and the stresses in the adjacent ground.

It is generally agreed with the Arching Theory and the Lateral Squeezing Theory the final stress state in the soil-bentonite is less than geostatic.

Terzaghi (1943), states that arching is the “transfer of pressure from a yielding mass of soil onto adjoining stationary parts” and that arching is one of the most common phenomena of soil behavior. Evans et al. (1995) applied arching theory to soil-bentonite cutoff walls and presented that the vertical stress in the soil-bentonite wall is given as a function of trench width, unit weight of the soil-bentonite, lateral earth pressure coefficient of the soil-bentonite, and interface friction between the soil-bentonite and the trench wall.

Lateral squeezing theory (Filz 1995) is an alternative to arching for predicting in situ stresses in soil-bentonite walls. In lateral squeezing, it is assumed that the trench walls can deform and that the amount and direction of movement influence the stresses in the soil-bentonite.

Evans et al. (1995) performed in situ tests and laboratory tests on a 4 year old wall, a 10 year wall, and a newly constructed soil-bentonite wall. Their results generally indicate that in situ stresses in the trench are low and are less than geostatic.

2.1.4 Deformations of Soil-Bentonite Cutoff Walls and Adjacent Ground

Deformations due to excavation of slurry filled trenches and consolidation of soil-bentonite cutoff walls shall be important design considerations since damages to adjacent buildings have been reported (Filz 1996).

Vertical deformations of soil-bentonite walls due to consolidation are reported for several case histories. Khoury et al. (1992) presented settlement versus time data from a soil-bentonite cutoff wall built in Manasquan dam. Some portions of the wall were 3 feet wide and other portions were 5 feet wide. The soil-bentonite wall was constructed in 2 stages. The lower stage was constructed when the dam reached 45 feet in height. The upper stage was constructed when the dam reached 55 feet in height. The upper stage was keyed into the lower stage by at least 3 feet. Vertical deformations with time were measured in the soil-bentonite trench using settlement plates. The lower stage was an average of 56 feet deep and underwent most of its settlement in 1-2 months. The upper stage was an average of 18 feet deep and experienced most of its settlement in about 2 weeks. The 3 foot section experienced a total of 3-4% vertical strain. The 5 foot section experienced a total of 7-9% vertical strain.

Engemoen and Hensley (1986) reported that a soil-bentonite cutoff wall at Calamus dam underwent 0.1% vertical strain, which occurred in one month. The cutoff wall was up to 110 feet deep with widths from 3 to 5 feet.

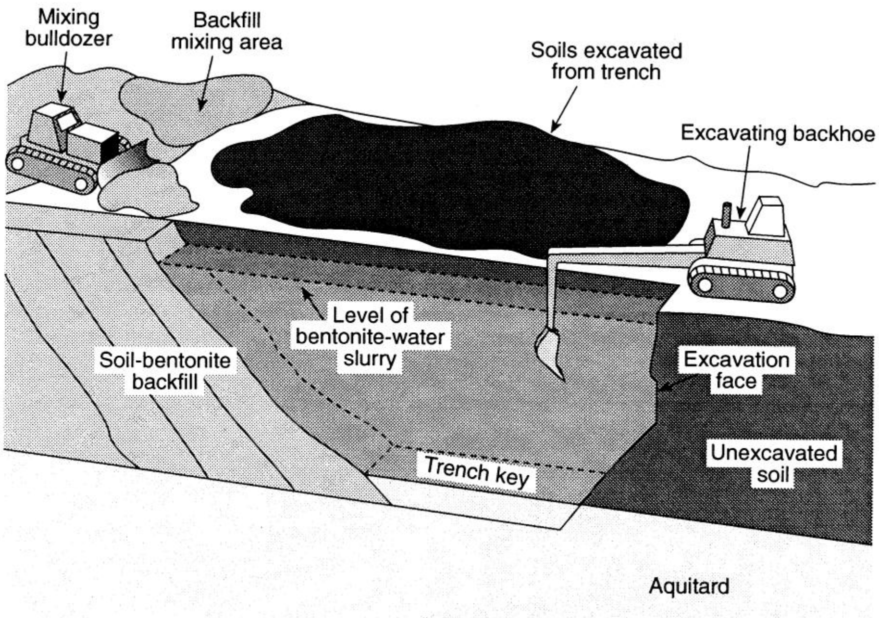


Figure 2.1 - Soil-Bentonite Cutoff Wall Construction Process

Source : *Barrier (*1995)

2.2 Soil-Cement-Bentonite (SCB) Cutoff Walls

Soil-Cement-Bentonite (SCB) slurry walls are an adaption of traditional soil or cement based walls (Ryan 1984). Fundamentally SCB wall is soil-bentonite slurry wall with cement added to the backfill (less than 10%). The benefit of slurry wall is that it is similar to the cement-bentonite wall in strength and to the soil-bentonite wall in hydraulic conductivity (Rumer at al. 1996).

2.2.1 Construction Method for Soil-Cement-Bentonite Slurry Walls

SCB walls are constructed following the same method as for SB slurry walls. The vertical narrow trench typically 2 to 5 feet wide is excavated by using a long reach excavator up to the relatively impermeable layer. The trench wall is supported by bentonite slurry (Figure 2.6). While the trench excavation is going on the backfill preparation is also done using the excavated/ borrowed soil, bentonite slurry/powder and cement slurry/powder according to the laboratory design mix. The trench backfilling is started with thoroughly blended materials of above when the trench excavation reaches considerable progress from its starting point. The slump down mixed material forms a slope in the trench by displacing the liquid slurry forward (Figure 2.7).

Figure 2.6 – Construction of Slurry Wall

Figure 2.7 – Slumping down the backfill

The backfill slope of SCB is usually in the range of 3 to 6:1 (horizontal to vertical), which is much steeper than SB backfill slopes. The backfill slope of SCB changes daily during the work, as the SCB hardens (Ryan & Day 2003).

The distance between the excavation point and the backfill operation point shall be maintained relatively constant, so that the two operations (excavation and backfilling) proceed at same rate.

Since the slope of the backfill is steeper with SCB, the amount of trench open at one time is reduced, providing greater trench stability than with SB (Ryan & Day 2003).

The backfill mixing process is carried out near the trench in an enclosed mixing area (Figure 2.8) and it gives distinct advantages, since transporting the backfill creates a delay in placement and additional costs. To perform the proportioning, mixing and placing the hydraulic excavators are commonly used. In addition to the excavators mixing boxes, mixing pits are often used to control material proportions.

Figure 2.8 – Mixing Process of Backfill

One method for achieving better quality during mixing is to add the cement in the form of a pre-mixed grout because the liquid grout is much easier to thoroughly mix with soil than dry cement and may have other technical advantages (Ryan & Day 2003). A typical grout plant for preparing cement slurry is shown in Figure 2.9.

Figure 2.9 – Cement slurry preparing grout plant

2.2.2 Properties of Soil Cement Bentonite (SCB) Backfill

Adding the cement grout to the backfill generally means a higher permeability that could be obtained with the same material without the cement. This is because the Portland cement interferes with the bentonite and prevents it from achieving its full swelling potential. Typical permeabilities for SCB backfill are in the range of 10^{-7} cm/sec.

SCB is stronger and more impermeable than cement-bentonite (CB) grout, but flexible enough to allow for deformations and usually less costly. Unlike CB, SCB permeability remains relatively unchanged over longer time intervals. SCB strength does continue to improve over time.

Strength of Soil-Cement-Bentonite (SCB) Walls

For projects where a moderate strength and a low permeability is needed, SCB can be an economical solution. Minimum strength specified for SCB walls is most typically in the range of 15 –100 psi (100 to 700 kPa), with the greater number of recent projects using a minimum of about 30 psi (200 kPa) at 28 days. This lower limit is probably in excess of actual project requirements for most installations.

There are numerous factors that should be considered by the designer in setting minimum (and maximum) strengths for SCB walls.

These include:

- The cost of cement that rises in almost direct proportion to the specified minimum strength.
- The addition of excessive cement may create joints in the backfill or decrease the flexibility of the wall under load, potentially leading to cracks caused by crushing, shaking or shear type of loadings.
- The long-term potential increases in strength over time.
- The negative effect that cement has on wall permeability, leading to greater flow through quantities than would be likely with an SB wall.
- The variability of the test results and the difficulty in accurately sampling and testing these lower strength materials.

Data from three actual projects were presented by Ryan & Day to illustrate the properties of SCB backfill and strength data summarized below for each project.

Dyke Cutoff Project- This project requires the sealing of the foundation of a long earthen dike. The objective was to find a feasible SCB design mix and basic mix methodology to meet a design spec of 30 to 300 psi (200 to 2100 kPa) at 28 days for UCS. The 28 days unconfined compressive strength values plotted against cement % as illustrated in Figure 2.10.

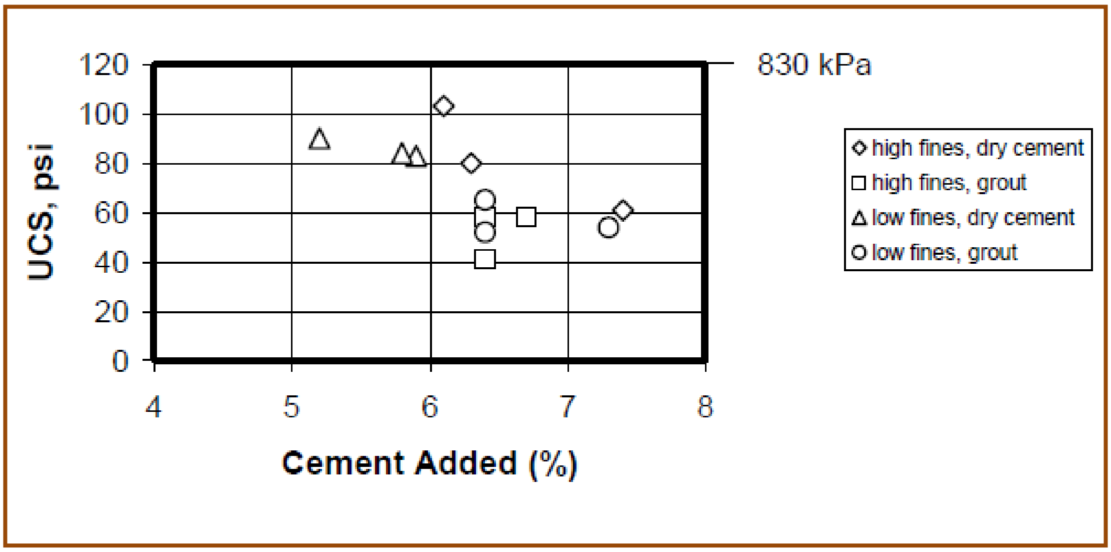


Figure 2.10 – 28 days Unconfined Compressive Strength Vs Cement added
Source: Ryan & Day 2003

All of the mixes met the strength specification. The interesting thing to note is that, those with cement mixed in as grout generally had a lower strength than those where the cement was added dry. (Other SCB projects have shown exactly the opposite trend (Zamojsky et al, 1995)). The mixes were cast with soil from two borings, one with high fines content, 57% and the other with low fines content, 12%. There was not a significant impact on strength based on fines content.

Mine Barrier Project - In the second project, only one test has run pre-construction to assess the mix design. The minimum strength requirement is 15 psi (103 kPa). The cement content selected was 3%. In this case, the SCB has selected to seal fractured rock and collapsed mine workings to stop the movement of black damp mine gas. Field results from five field samples were in the range of 15-20 psi (103-138 kPa), while the single preconstruction test gave a result of 27 psi (186 kPa).

Embankment Cutoff Project - The third project has done in two phases. The unconfined compressive strength specification for Phase 1 of the embankment project was 15 psi (103 kPa) (Figure 2.11) and for Phase 2 of the embankment, 30 psi (207 kPa) (Figure 2.12).

The cement content for Phase 1 was 3% and it was added dry. For Phase 2 of the same project, the cement content was 5% and it was added in the form of a pre-mixed grout.

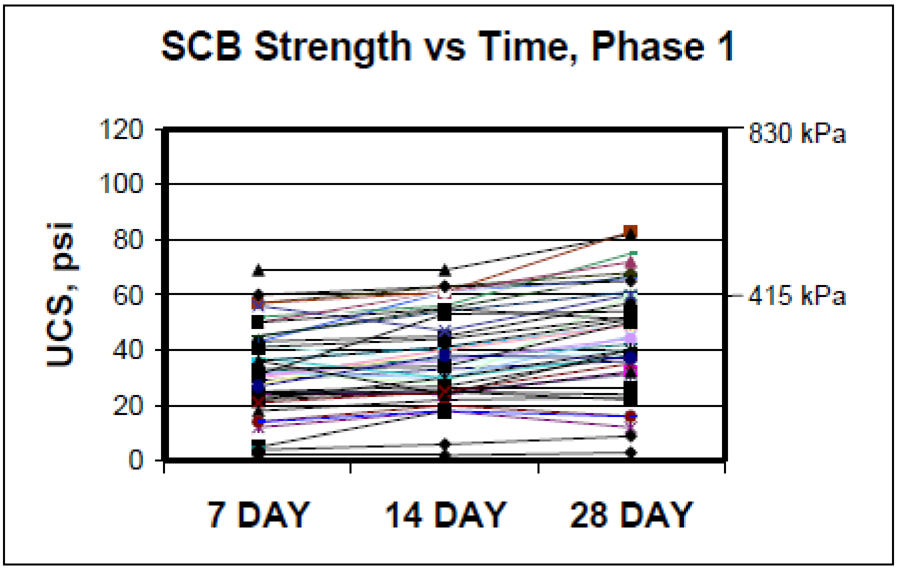


Figure 2.11 - Influence of Time on Strength, Cement Added Dry, Phase 1
Source : Ryan & Day 2003

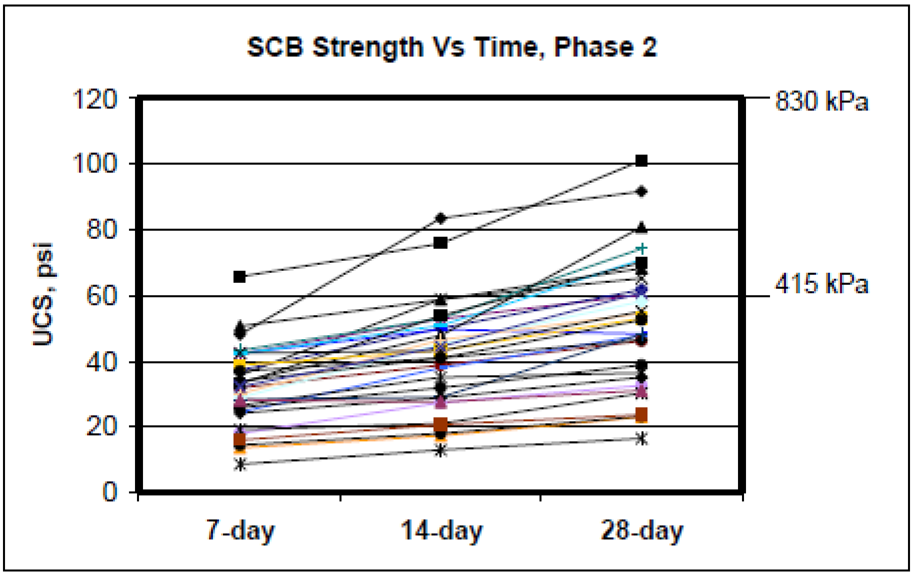


Figure 2.12 - Influence of Time on Strength, Cement Added as a Grout, Phase 2
Source : Ryan & Day 2003

There is a fairly consistent trend in the increase of strength over time. On the average, 7- day results are approximately 60% of the 28-day results and the 14-day results are about 80% of the 28-day results.

<u>Permeability of Soil-Cement-Bentonite (SCB) Walls</u>

The permeability (or hydraulic conductivity) of an SCB backfill is the result of complex interactions between the various components of the mix. Clearly, Portland cement interferes with the normal ability of a soil-bentonite blend to achieve very low permeability. A typical soil-bentonite wall specification will require a permeability of 1 x 10^{-7} cm/sec, and this is a level that is relatively easily attainable on almost every project. With SCB backfill, a specification requirement of 5 x 10^{-7} cm/sec is typically difficult to meet and may require special construction procedures and mix components to attain.

Factors that need to be considered when specifying a SCB mix or when trying to design a mix, to achieve specified properties were cited by Ryan & Day in 2003 and are as follows;

- The addition of Portland cement to the wall has a negative effect on permeability t-hat it generally increases as the cement quantity increases. Not only does Portland cement chemically affect the ability of bentonite to "swell" and retain water, but it also requires water to be added to wet the mixture to achieve slumpable material for placement. More water leads to a less dense and more porous backfill as it sets.

- Increasing bentonite quantity will not necessarily have the same beneficial effect that it would in a normal SB backfill. Portland cement interferes with its efficiency and the additional bentonite again requires more water to wet the mix for placement.

- Additives may be helpful in reducing permeability, but they also complicate the construction process and add to the cost. Additives that have been used include lignosulfonate retarder and thinners that are used to prepare concentrated bentonite slurries for addition.

- There is some evidence that a minimum amount of fines may be beneficial in achieving optimal performance. A minimum of 10% plastic fines is recommended for a well-proportioned SCB mixture. On the other hand, excessive fines may require additional water in the form of bentonite slurry for wetting to achieve placement slump and again may be less dense.

- Adding cement in the form of a grout may provide a benefit in the form of more consistent results. Again this needs to be assessed on a project-by-project basis.

Permeability results of above indicated three actual projects by Ryan & Day are summarized below for each project.

Dyke Cutoff Project - All of the mixes have not met the 5x 10^{-7} cm/sec permeability specification. Certain minimum fines quantity will be necessary to consistently meet the permeability specification since almost all of the tests using the soil sample with 12% fines failed to meet the requirement (Ryan & Day).

The plotted data as in Figure 2.13 and bentonite has added as a dry additive (as a per cent of the dry weight of soil).

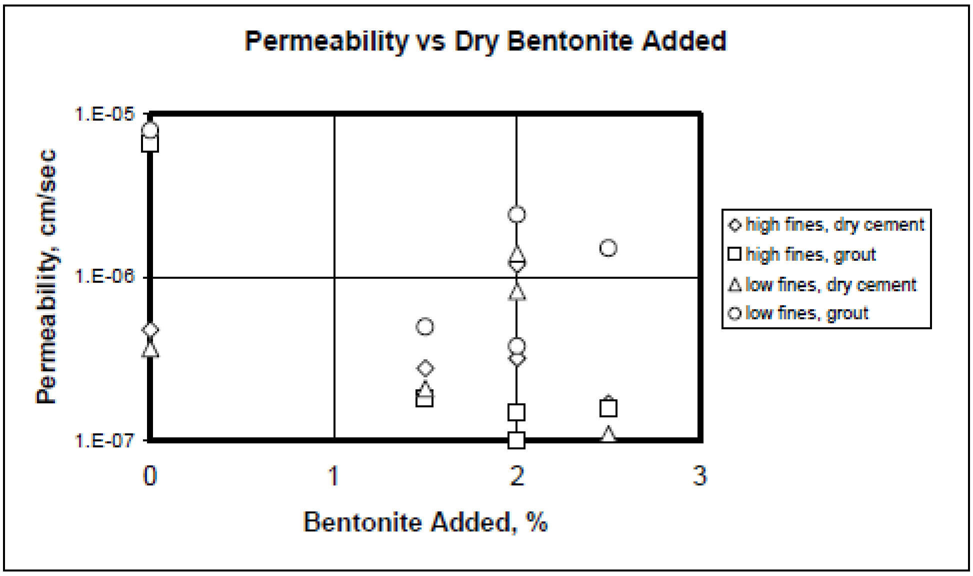


Figure 2.13 - Influence of Additional Dry Bentonite on Permeability
Source : Ryan & Day 2003

According to the plotted data above, cement added as grout seems to provide a mix with a more consistent low permeability. Ryan & Day stated that it may be due to the grout being easier to mix and therefore, more homogeneous or due to the fact that pre-hydrating the cement may decrease the negative effect it has on the bentonite.

Mine Barrier Project - the mine gas barrier, was for a much smaller project and the specification was 1 x 10^{-6} cm/sec maximum permeability. The SCB mix was helped by relatively high fines content, 40%. For this project only one pre-job test has run and only five field samples have tested , all passing.

Embankment Cutoff Project - The embankment has done in two phases and the specification for both phases was for a maximum permeability of 5 x 10^{-7} cm/sec. Dry bentonite added for phase 1 was 1.8% weight of dry soil. The Phase 1 results, presented on Figure 2.14, show a slight trend for improvement of permeability measurements with time. There is considerable variability which is typical of this material and which is partly caused by sampling problems. In this case, there are a significant number of tests that fell above the specified minimum. In some cases, these samples were retested using archived samples and subsequently passed (Ryan & Day).

A small portion of this project was dug up and remixed. It turned out that the bad section passed through a zone with little fines and the addition rate of dry bentonite had to be increased.

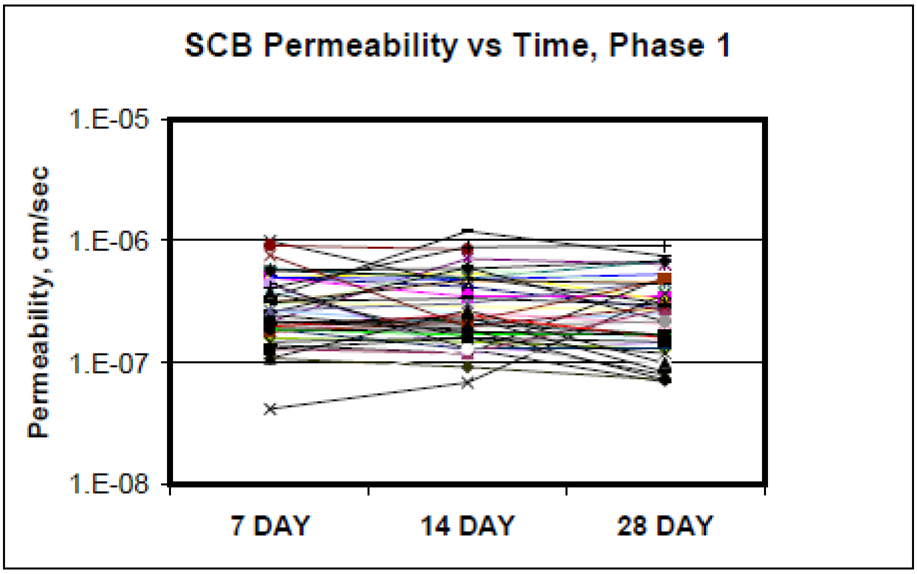


Figure 2.14 – Permeability variation with time - Phase 1
Source : Ryan & Day 2003

Backfill for the second phase had no dry bentonite (bentonite by sluicing only). Cement has added in the form of a pre-mixed grout and mixing has done in a mixing box. The field data for this second phase, shown in Figure 2.15, are actually more consistent than those from the first phase.

Almost all of the data have passed the specified test requirement and the points that failed have all supplemented by archived samples that passed. Since SCB properties improve with time, the archiving of samples is always important for a project of this type (Ryan & Day).

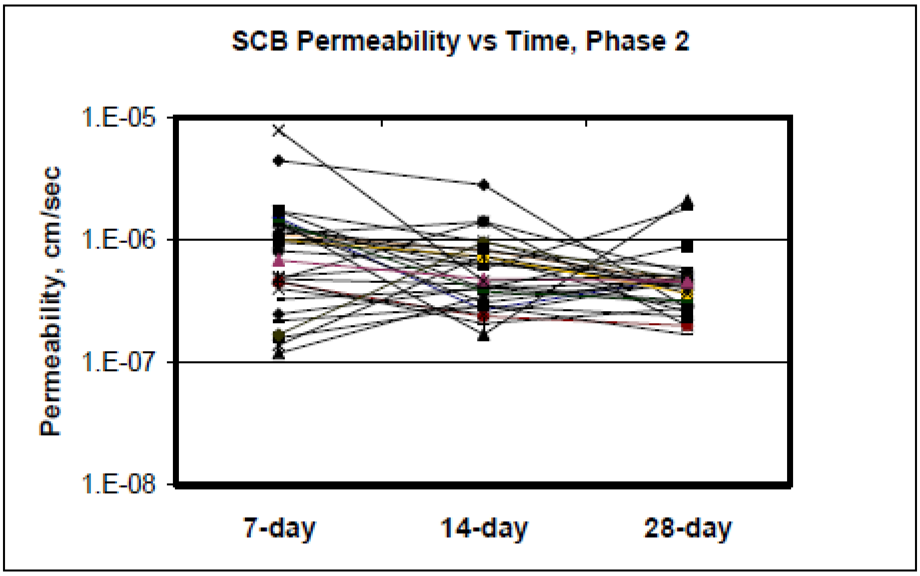


Figure 2.15 – Permeability variation with time - Phase 2
Source : Ryan & Day 2003

2.2.3. Documented Detailed Case Studies on Soil-Cement-Bentonite (SCB) Slurry Cutoff Walls

The published case studies for Soil-Cement-Bentonite (SCB) slurry cutoff walls are very few compared to the Soil-Bentonite (SB) slurry cutoff walls. In this section two case studies are summarized to illustrate the performance of the constructed SCB slurry walls.

<u>CASE STUDY 01</u>

CONSTRUCTION AND IN-SITU HYDRAULIC CONDUCTIVITY EVALUATION OF A DEEP SOIL-CEMENT-BENTONITE CUTOFF WALL BY D.G. RUFFING AND J.C. EVANS IN 2014.

A deep Soil-Cement-Bentonite (SCB) slurry trench cutoff wall was constructed around the perimeter of a deep excavation to reduce long-term dewatering costs associated with construction of a hydroelectric power plant adjacent to the Ohio River of Smithland city in Livingston County, Kentucky, United States in 2010. At the time of its installation, the Smithland cutoff wall was the deepest conventional Soil-Cement-Bentonite cutoff wall.

A large scale estimation of the *in-situ k* of the wall was conducted by utilizing steady state groundwater flow measurements from the dewatering system coupled with information on the wall thickness and water levels inside and outside of the wall. The *in-situ k* was compared to laboratory *k* values measured for specimens prepared from grab samples of the as-mixed SCB backfill.

The project site is located immediately adjacent to the Ohio River in Smithland, KY, US. The site view is as in Figure 2.16.

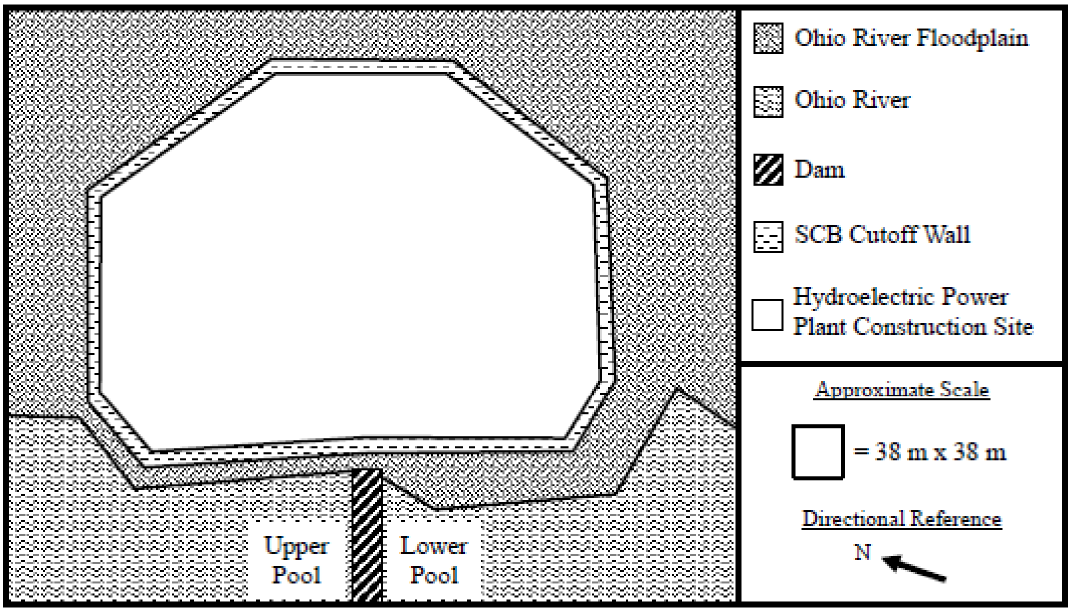


Figure 2.16 – Plan View of the SCB cutoff wall in Smithland

The overburden soil consistent with alternating river deposits of varying classification and grain size. The project designers assumed that the overburden soils approximately 47 m (155 ft) down from the ground surface were highly permeable. These materials were assigned a horizontal hydraulic conductivity of $1x10^{-1}$ cm/s due to the presence of cemented sand and gravel layers. The limestone bedrock underlies the overburden soils down to 100 m is karstic in nature. The design hydraulic conductivity value for limestone layer was $3x10^{-4}$ cm/s and underlying second bedrock was $1x10^{-5}$ cm/s.

The design objectives of the SCB slurry cutoff wall was 207 kPa in strength and $1x10^{-6}$ cm/s in hydraulic permeability.

A preconstruction bench scale study was conducted to assess the feasibility of soil-cement-bentonite mixture that would meet the project objectives. Two site soil composites, Composite 1 and Composite 2, were created using soils collected in five borings along the cutoff wall alignment and are as in Table 2.3.

Table 2.3 - Preconstruction Bench Scale Soil Index Test Results

Composite ID	**Lab Description**	**Moisture Content (%)**	**Fines Content (%)**
Composite 1	Silty Sand,Non-Plastic Fines	16.0	29.5
Composite 2	Silty Sand,Non-Plastic Fines	11.5	19.0

Two Portland cement addition rates were tested on the two site soil composites for a total of four SCB mixes. One to one (by weight) cement to water grout and 6% bentonite to water (by weight) slurry prepared to maintain the water content of the mixture. The compositions of the four SCB mixes are summarized in Table 2.4.

Table 2.4 - Preconstruction Bench Scale Soil Index Test Results

Mix ID	**Soil Composite**	**Bentonite (%)**	**Cement (%)**
S-1	Composite 1	0.9	5
S-2	Composite 1	0.8	7
S-3	Composite 2	0.6	5
S-4	Composite 2	0.5	7

After 14 days of curing, unconfined compression and hydraulic conductivity (*k)* testing were performed and the results are presented on Table 2.5.

Table 2.5 - Hydraulic Conductivity and UCS of Preconstruction Bench Scale Study Mixes

Mix ID	**14 day UCS**	**14 day Hydraulic Conductivity, k**
S-1	172 kPa	4.6×10^{-7} cm/s
S-2	193 kPa	4.7×10^{-7} cm/s
S-3	400 kPa	6.7×10^{-7} cm/s
S-4	470 kPa	1.2×10^{-6} cm/s

The mixes created from Composite 1 exhibited a lower *k* and the mixes created from Composite 2 exhibited a higher strength. The higher cement content produced higher strength results compared to the lower cement content for each soil composite. The only result that did not entirely fit with the others was the *k* of mix S-4 which was prepared at the high cement content and mixed with a composite sample representing the lower fines content. This may be due to an anomalous structure of the specimen with higher cement content combined with the lower fines content.

With the past experience of the designers SCB backfill strength can increase by approximately 50% and the *k* can be decreased by approximately 100% from 14 to 28 days (Ruffing & Evans in 2014). Considering the 14 day results designers believed that S-1 & S-3 mixes could meet the design objectives as in Table 2.6.

Table 2.6 - Expected Hydraulic Conductivity and UCS of Recommended Mix

Mix No	**28 day UCS**	**28 day Hydraulic Conductivity , *k***
Composite 1 (S1)	240 kPa	3×10^{-7} cm/s
Composite 2 (S3)	600 kPa	4×10^{-7} cm/s

During the excavation process of the trench an excavator fitted with a specialty boom and long stick was used up to 27 m below the ground surface. The rest was completed using crane mounted hydraulic and mechanical clamshell buckets digging primary and secondary panels. The average wall depth was 47 m (155 ft) and the maximum wall depth was 56 m (185ft). Figure 2.17 shows a photograph of the long stick excavator and clamshells excavating the cutoff wall.

Figure 2.17 - Photograph of Clamshells (foreground) and Long Stick Excavator (background) excavating the Smithland Cutoff Wall

Initial SCB backfill placement was completed using a custom built tremie pipe. After the “head” of the backfill reached the surface, the SCB backfill was pushed into the trench using a small hydraulic excavator.

For this project, SCB backfill samples were taken immediately after the backfill was mixed and prior to placement in the trench at a frequency of 1 sample for every 760 m^3 of backfill placed. Permeability and UCS tests were conducted after 7 or 14 and 28 days of curing with the 7 or 14 day results used as preliminary indicators and the 28 day results used to determine acceptance.

Thirty nine grab sample locations were tested for the Smithland cutoff wall. The average, maximum, and minimum UCS and *k* results from tests conducted on 28 day old specimens are presented on Table 2.7.

Table 2.7 - Avg., Max., and Min. Hydraulic Conductivity and UCS of Grab Samples

	28 day UCS	**28 day Hydraulic Conductivity , *k***
Average	365	2.2×10^{-7} cm/s
Maximum	738	8.7×10^{-7} cm/s
Minimum	186	7.8×10^{-8} cm/s

The results of the *k* and UCS testing on grab samples were consistent with the results of preconstruction bench scale testing using site soils and performing better than the design objectives of 207 kPa and 1×10^{-6} cm/s.

Estimation of the *in-situ k* of the cutoff wall was done assuming steady flow and using Darcy's law with site-specific parameters in Table 2.8.

$$q = kiA$$

$$q = k\left(\frac{(WL_o - WL_i)}{W}\right)(A_w)$$

Table 2.8 - Observed and Calculated Average Parameters for the Smithland Cutoff Wall

Parameter	*Range*
Wall Width, W	0.91 m
Wall Length, L	1161.9 m
El. of "Rock"	57.5 m
Average Wall Height, H	46.9 m
Water Level Outside (elevation), WL_o	96.0 m
Water Level Inside (elevation at steady state pumping), WL_i	56.1 m
Wet Height, $H_w = WL_o$ – El. of "Rock"	38.5 m
Pumping Rate, q	7.6×10^5 cm^3 / s (avg)
Wetted Area, $A_w = L \times H_w$	4.5×10^8 cm^2

Case 1 - To obtain an estimate of the upper limit of the wall *k*, all flow entering the system was assumed to be entering through the wall and not under the wall. This assumption is very conservative (unrealistic) given the karstic nature of the bedrock underlying this site and the understanding that high permeability "windows" are inevitable beneath the wall. The maximum *in*-situ *k* for the barrier of 3.8×10^{-5} cm/s was calculated.

Case 2 - The infiltrating groundwater is a combination of flow through the wall, flow through overburden soil windows beneath the wall and flow beneath the wall through the bedrock. Assume that the cutoff wall *k* is equal to the measured *k* from the laboratory tests on the grab samples and use this assumption in above equation to calculate the flow through the wall.

This calculation yields that flow through the wall is slightly more than 0.5% of the total flow. It reveals that the wall is seated on the bedrock more or less than 96% of its length. The flow through the underlying bedrock makes up approximately 11.8% and the flow through the soil windows makes up approximately 87.7% of the total flow entering the system.

Table 2.9 shows a summary of the estimated *k* from the two cases presented above.

Based on these analyses, flow through the cutoff wall is probably very small in comparison to the flow beneath the wall.

Table 2.9 - Summary of Estimated Cutoff Wall *k* from Dewatering Data

Case No.	**Flow through Wall (%)**	**Flow through Overburden Soil Windows (%)**	**Flow through Underlying Bedrock (%)**	**Estimated Wall *k* (cm/s)**
Case 1	100	0	0	4×10^{-5}
Case 2	0.5	87.7	11.8	2×10^{-7}

CASE STUDY 02

CONSTRUCTION OF A SOIL CEMENT BENTONITE SLURRY WALL FOR A LEVEE STRENGTHENING PROGRAM BY LOUAY M. OWAIDAT ET AL. 1998

Background

The U.S. Army Corps of Engineers has constructed a soil-cement-bentonite slurry wall through the existing levee of the American River in Sacramento, California to improve stability by preventing seepage through and beneath the levee during flood stages when the river is high. Challenges to the barrier performance included achieving a maximum allowable hydraulic conductivity of 5 x10^{-7} cm/sec while having a minimum unconfined compressive strength of 15 psi. The slurry wall project map is shown in Figure 2.18.

The construction was done during the period of August to September, 1998 and within 9 weeks in a residential area with severe space limitations. Four large excavators capable of excavating to maximum depth of 26 m were utilized. In order to meet the tight schedule and performance requirement, the barrier wall backfill mix was designed to fulfill the specified 28 day hydraulic conductivity requirement by 7 days.

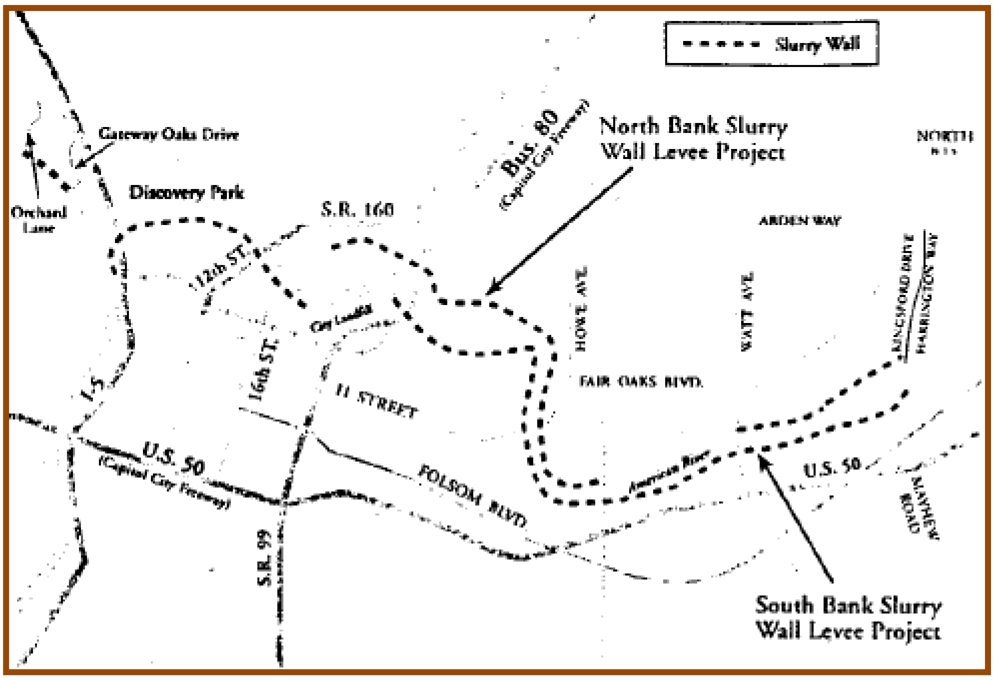


Figure 2.18 – Project map of slurry wall site in Sacramanto, California

Subsurface Details

The levee consists primarily of sandy to silty soil. Beneath the levee, the boring encountered layers of sandy, silty and clayey soil deposits of various thicknesses and at various depths. A gravel and cobble layer was encountered underlying these deposits along the levee. This gravel and cobble layer varied 5-40 ft in depth and 5-30 ft in thickness. It was believed that this gravel and cobble layers are serving as a channel for seepage flow toward the landside of the levee and cause seep and boil conditions on the land side ground surface. Clayey to sandy soil deposits were encountered beneath this more permeable layer as shown in Figure 2.19.

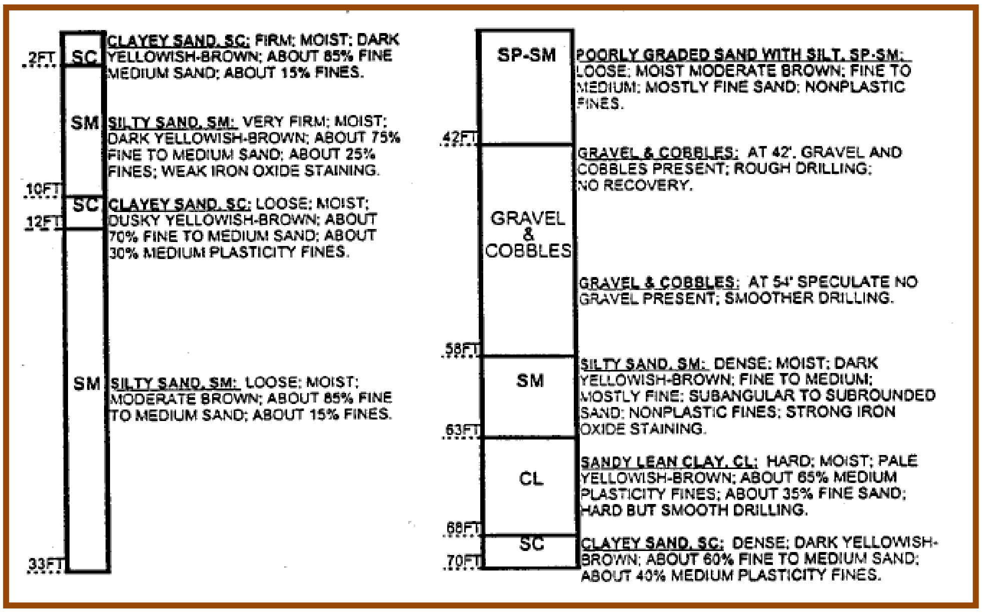


Figure 2.19 – Subsurface profile of slurry wall site in Sacramanto, California

Design and Construction

Potentially large hydraulic head, little shear strength in soil-bentonite (SB) and greater erosion resistance in soil-cement-bentonite (SCB) were the key points to select the SCB backfill for the barrier wall. The excavated soils were suitable and used after removing cobbles and blending fine and coarse materials.

Prior to the construction a laboratory mix design was conducted to predict soil-cement-bentonite performance and to determine material proportions for the soil-cement-bentonite mixture. The mix design utilized site soils, American river water, bentonite and cement. Two soil composites were prepared from the levee site and the amount of fines passing the No. 200 sieve was 41 % and 49 %. Table 2.10 shows the mix proportions of eight samples prepared for the testing. The bentonite slurry contained 5.4% bentonite by weight of water and the cement slurry contained 150% cement by weight of water were adapted. The samples were tested for hydraulic conductivity and unconfined compressive strength at 7, 14 and 28 days.

The hydraulic conductivity and unconfined compressive strength results are shown in Table 2.11.

Table 2.10 – Mix proportions of pre construction testing

Mix No:	***Grain size (% passing)***			***% Bentonite added by wt of soil***	***% Cement added by wt of soil***
	# 4	***# 30***	***# 200***		
1B	100	94	49	2.9	6
2B	100	94	49	2.5	8
3B	100	94	49	2.9	4
4B	100	94	49	3.3	8
5B	100	94	49	2.2	4
6B	100	94	41	2	4
7B	100	94	41	2	6
8B	100	94	41	2	8

Table 2.11 – Hydraulic Conductivity and Unconfined Compressive Strength Results

Mix No:	W/C %	Permeability @ 7 days (cm/sec)	Permeability @ 14 days (cm/sec)	Permeability @ 28 days (cm/sec)	Strength psi @ 7 days	Strength psi @ 14 days	Strength psi @ 28 days
1B	67.9	8.0×10^{-7}	8.7×10^{-7}	7.8×10^{-7}	10	12	15
2B	57.1	5.9×10^{-7}	6.8×10^{-7}	4.4×10^{-7}	13	22	25
3B	68.1	5.5×10^{-7}	6.6×10^{-7}	5.1×10^{-7}	9	11	14
4B	69.4	5.4×10^{-7}	7.0×10^{-7}	5.6×10^{-7}	8	14	17
5B	57.8	4.6×10^{-7}	5.6×10^{-7}	4.3×10^{-7}	10	15	18
6B	46.7	5.0×10^{-7}	5.5×10^{-7}	4.2×10^{-7}	11	17	18
7B	52.7	7.1×10^{-7}	5.1×10^{-7}	4.2×10^{-7}	18	23	28
8B	47.5	4.7×10^{-7}	2.6×10^{-7}	1.8×10^{-7}	25	32	42

The mix no 5B was selected and it was modified high concentration bentonite (11% by weight of water) instead of the 5.4 % bentonite slurry. The modified mix produced a permeability of 4.7 $x10^{-7}$ cm/sec and an unconfined compressive strength of 11 psi at 7 days.

During the construction slurry trench of 2.5 feet wide was excavated with the support of bentonite slurry. Usually the trenches are excavated deeper or keyed in to an acquiclude to form an impervious base. For this project sandy silt with gravel (ML), clayey sand (SC) acquicludes were found and wall was keyed 3 feet in to it.

The bentonite slurry (5-6% bentonite by wt of water) was produced at a batch plant using a jet shear mixture and transferred to the slurry pond prior to introduction in to the trench. The high concentration bentonite slurry (10-12 % bentonite by wt of water) and cement slurry were produced with high speed colloidal mixers. Soil-Cement-Bentonite backfill was mixed in a prepared earthen pond (backfill mixing pond) using a hydraulic excavator. A known volume and density of homogenized excavated soil was mixed with a known volume and density of high concentration bentonite slurry and cement slurry in accordance with the laboratory mix design proportions. The backfill was transported from the mixing pond using trucks to the open trench where it was placed using a small excavator. Backfill was initially placed using a ramp excavated in the soil on one end of the trench. Backfill in the trench formed a relatively flat slope of approximately 5:1 to 8:1 and the minimum distance of 50 feet between the toes of backfill and the excavation was maintained to maximize the stability of the trench.

Following the construction of soil-cement-bentonite slurry cutoff wall , a cap consisting of compacted impervious fill material was placed between the top of the slurry wall and the final grade of the levee.

During the quality control program 96 soil-cement-bentonite samples were tested for permeability and compression test at 7, 14 and 28 days. The test results plotted were consistent with the mix design results and exceeded the design criteria as shown in Figure 2.20 to Figure 2.23.

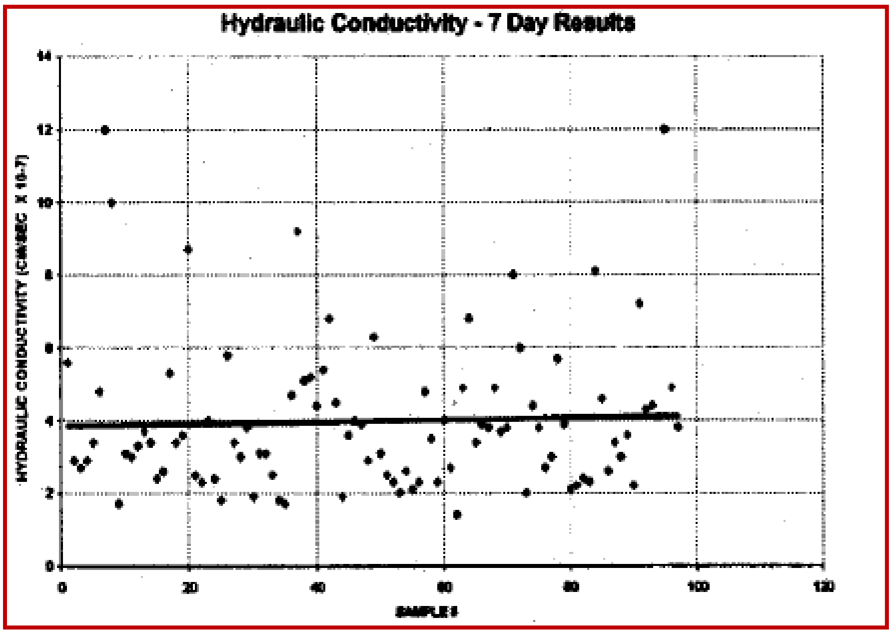


Figure 2.20 – Hydraulic Conductivity Results at 7 days

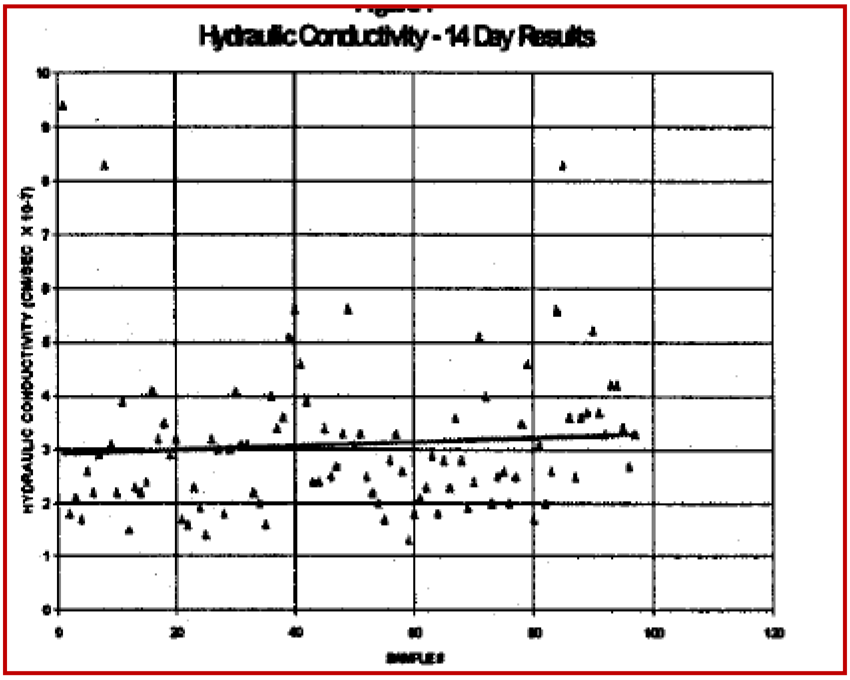


Figure 2.21 – Hydraulic Conductivity Results at 14 days

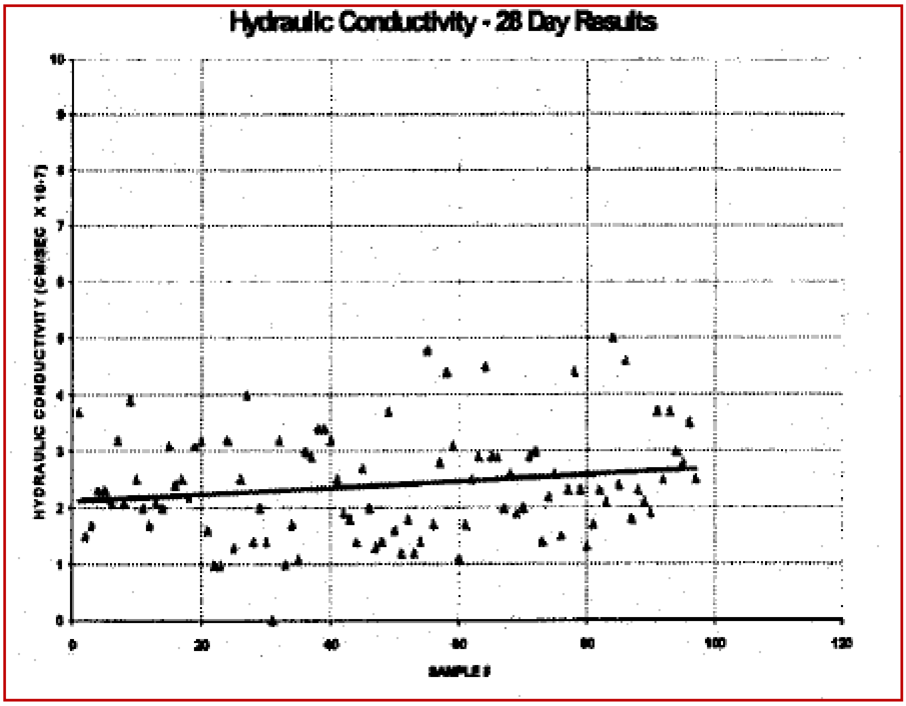


Figure 2.22 – Hydraulic Conductivity Results at 28 days

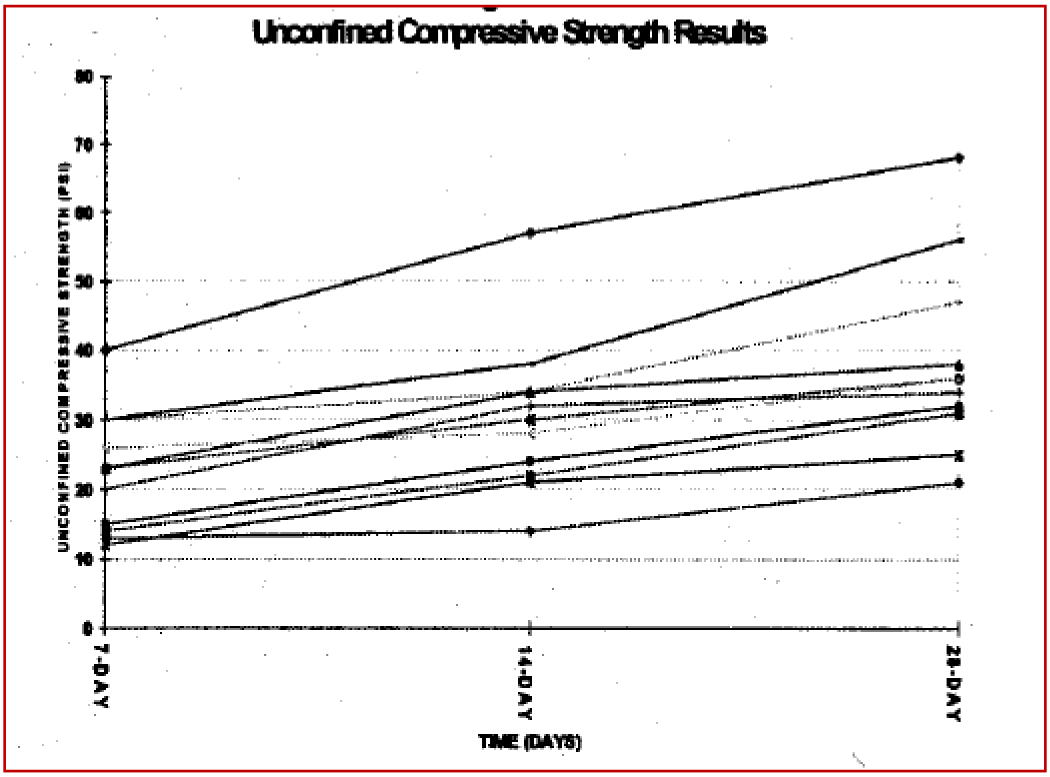


Figure 2.23 – Unconfined Compressive Strength results

3. Methodology

This chapter comprises the methodology followed to fulfill the objectives of the study. It can be summarized in to a flow chart as shown in Figure 3.1.

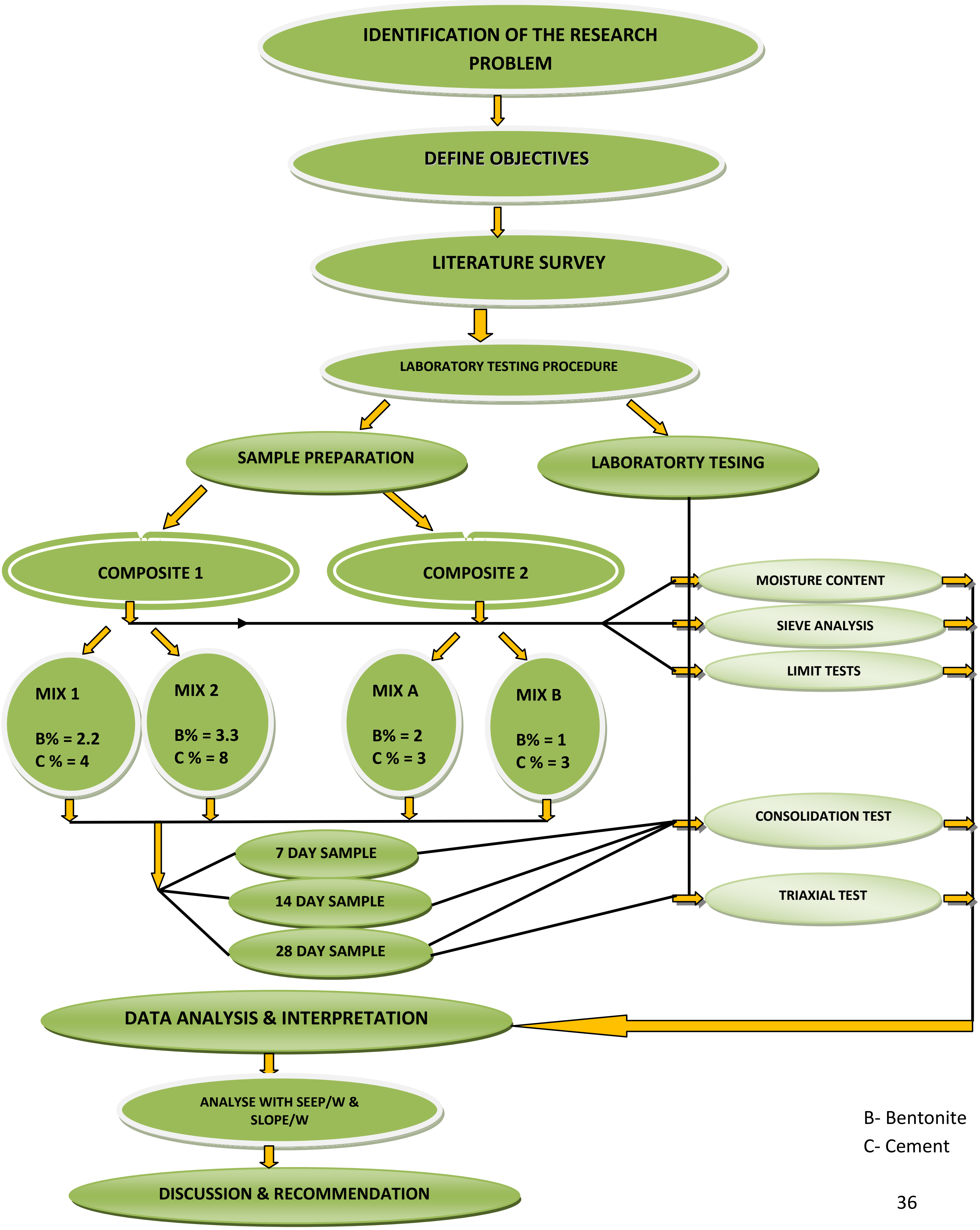


Figure 3.1 – Flow Chart Illustrating the Research Procedure

3.1 Identification of the Research problem related to the Vendrasan Dam

The Vendrasan dam, an ancient dam is located Trincomalee in eastern province of Sri Lanka. This tank bund has been reported for heavy seepage through the dam body and beneath the foundation well over 20 years. The tank details are as follows,

- Bund Length 700m
- Bund Height 15m
- Crest Width 4m

The dam could not be filled more than half of its capacity due to the downstream seepages which is badly affects the stability of the dam. Most of the downstream area was swampy. Rehabilitation works done in 1988 by Irrigation Department, the owner of the dam and there were no any improvement. They have placed a stabilizing fill in the toe which became again a swampy after sometimes. Figure 3.2 shows a flat plan drawn by Irrigation Department in 2005. It can clearly observe that sink holes and boggy area in the critical section of CH 250-690.

In 2006 also Irrigation Department has done a clay grouting process which became unsuccessful after sometimes. Dam Safety & Water Resources planning project (DSWRPP) in 2011 conducted a comprehensive study on the dam and the project consultants (Poyry) came out with durable solutions, place a secant pile wall along the critical section. But finally during the dam total rehabilitation work in 2012, it was again done a grouting process to mitigate seepage of this critical section as the owner of the dam disagrees on consultants' solution. Ultimate result is seepage through the dam remain as an unsolved issue.

3.1.1 Geology of the dam site

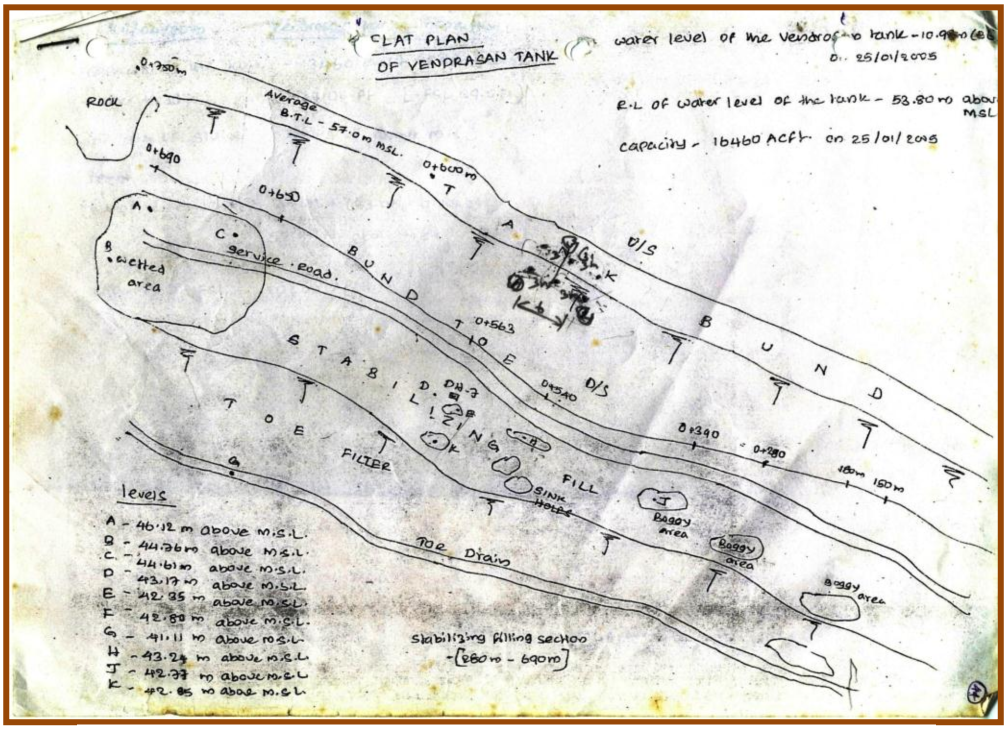


Figure 3.2 – Flat plan of D/S of Vendrasan dam drawn in 2005
Source: Geology Branch of ID

Eight (8) bore holes were carried out by the Geology Division of Irrigation Department in 2005 where more sink holes could be observed and Figure 3.3 shows the drill hole location map.

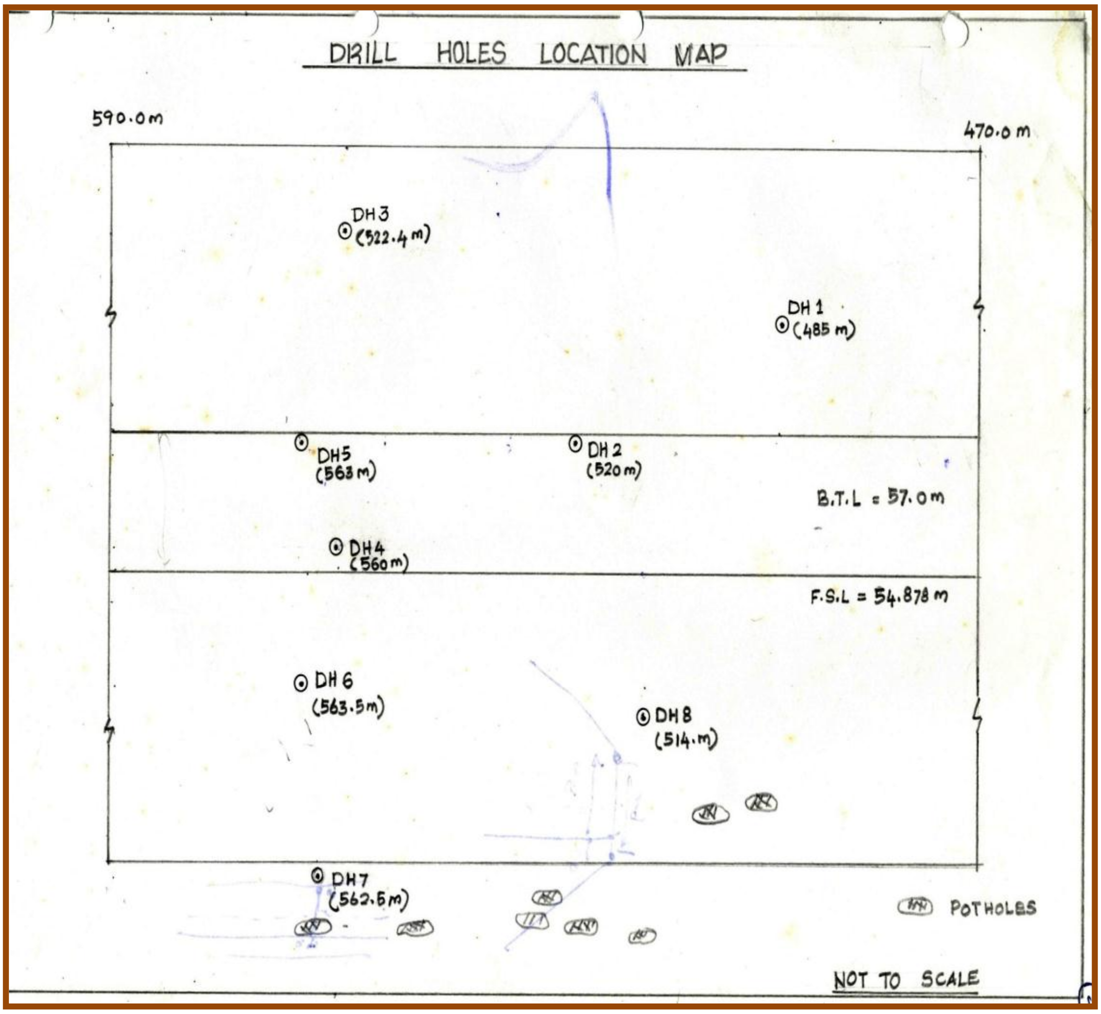


Figure 3.3– Drill Hole location map of Vendrasan dam in 2005
Source: Geology Branch of ID

It was revealed from the details of the borehole logs in Figure 4.3 to 4.7, the dam body consists with unsuitable materials with poor impermeability. The sand layers in varying thicknesses may serve as the seepage channel in the dam body within the seepage section.

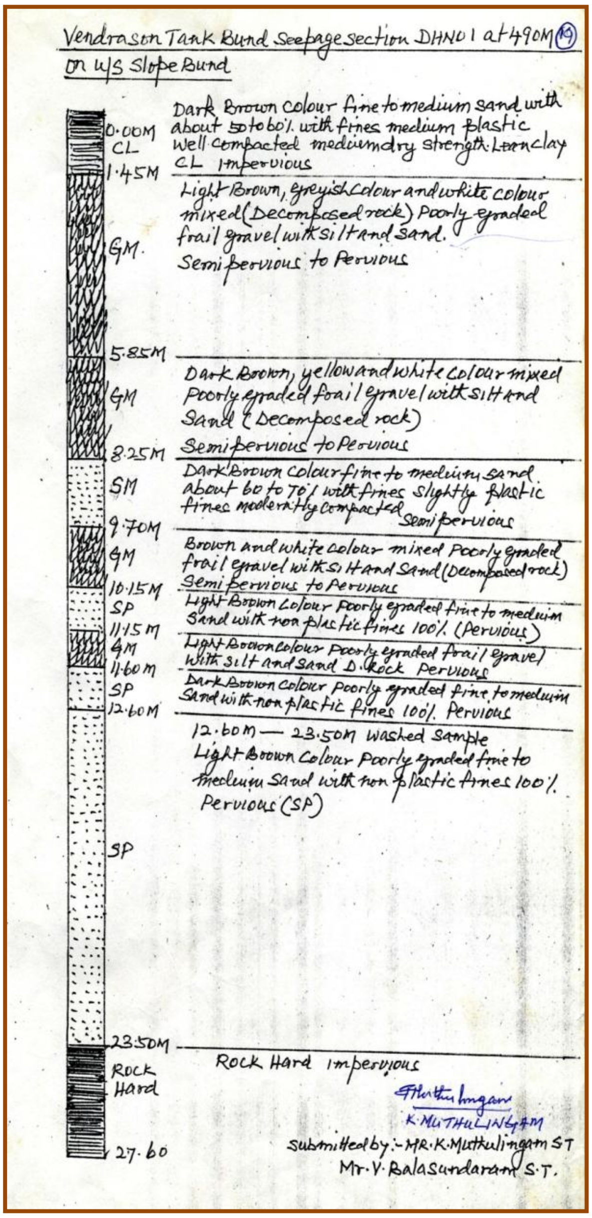


Figure 3.4 – BH No. 01 of Vendrasan Dam

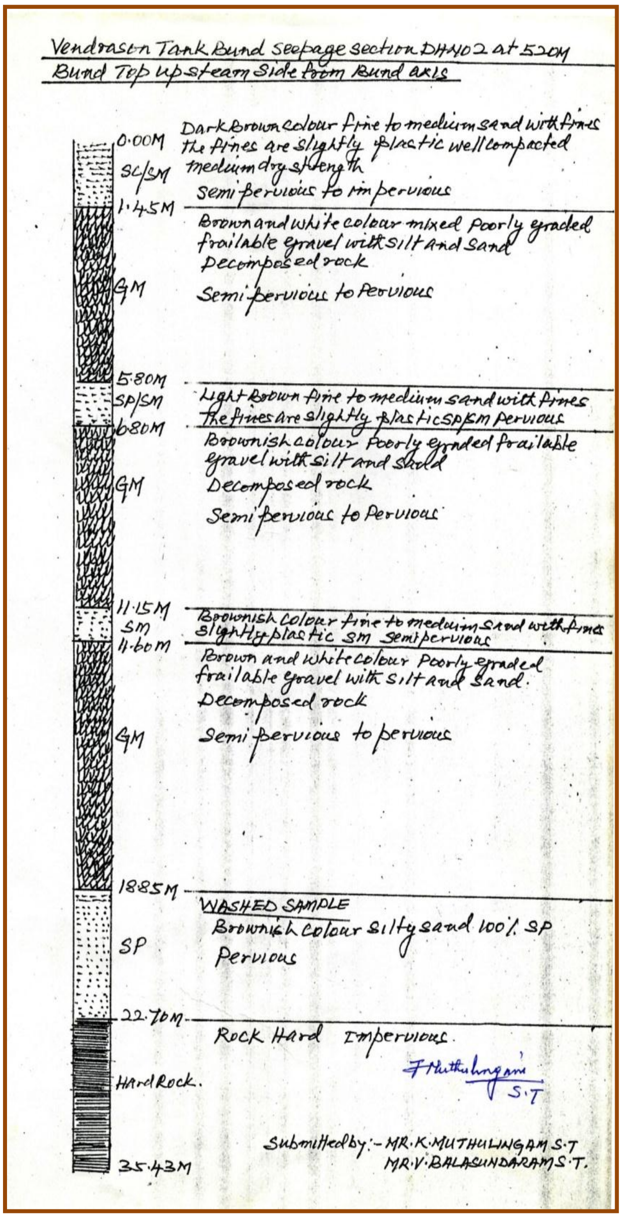


Figure 3.5 – BH No. 02 of Vendrasan Dam

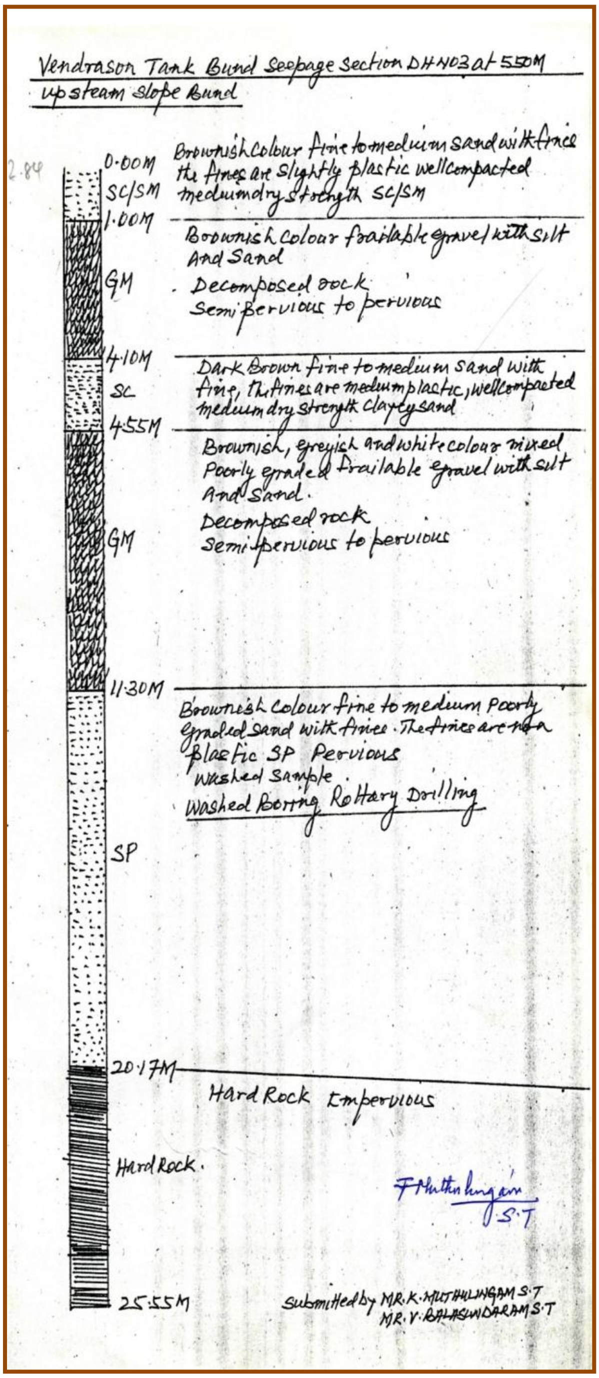


Figure 3.6 – BH No. 03 of Vendrasan Dam

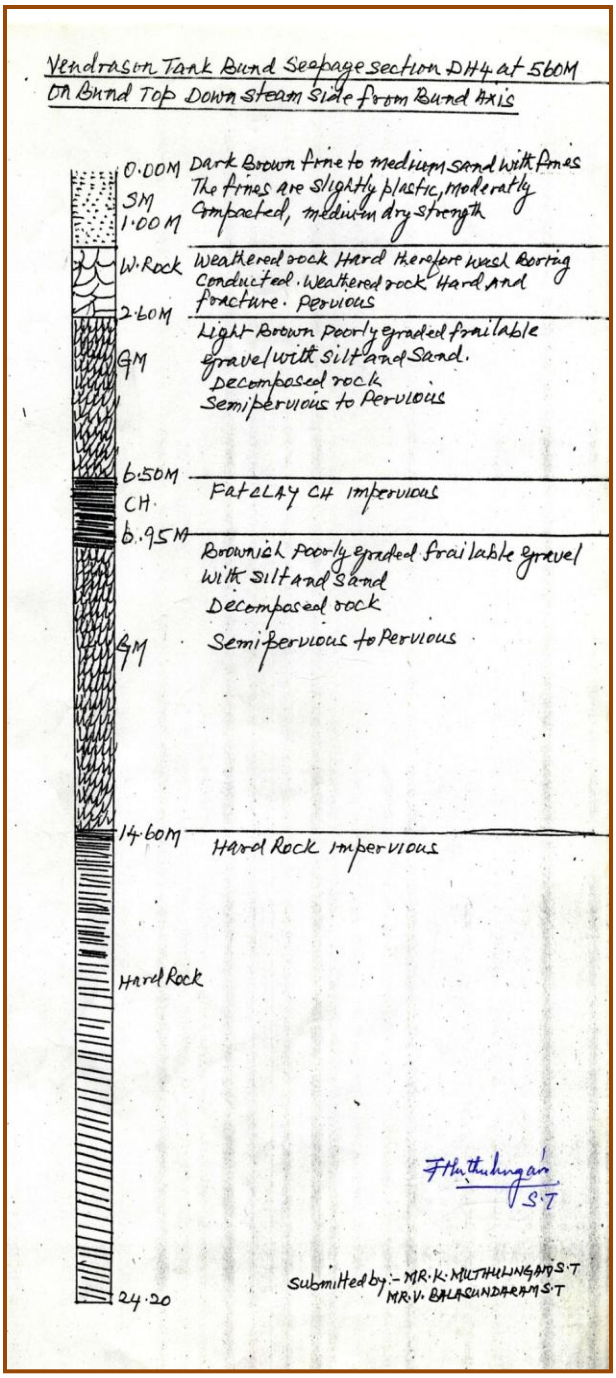
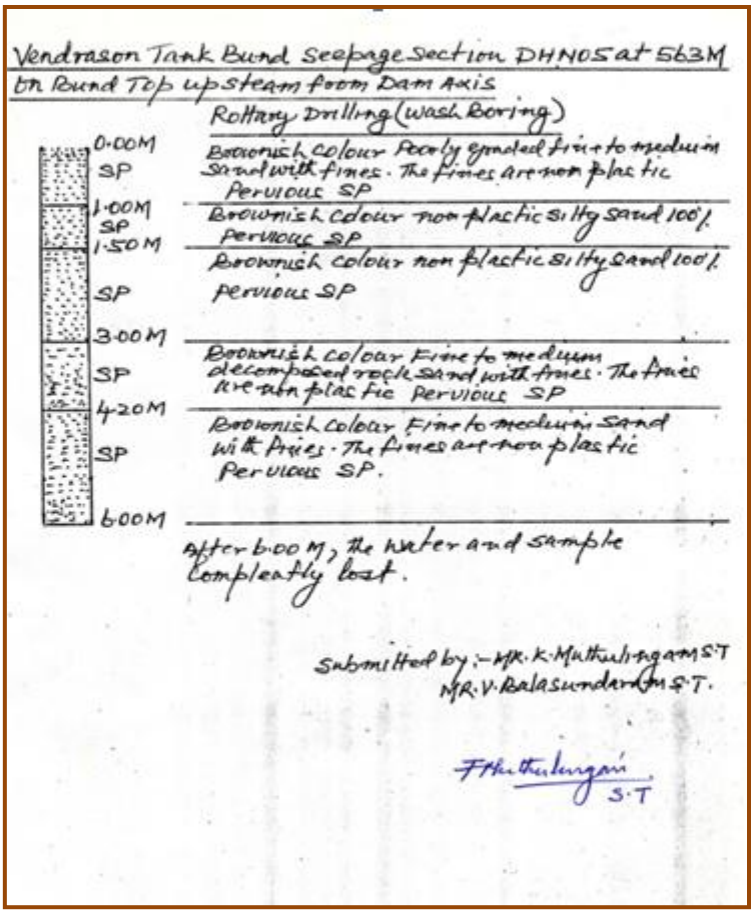


Figure 3.7 – BH No. 04 of Vendrasan Dam



Vendrason Tank Bund Seepage Section DH No 5 at 563M
on Bund Top upstream from Dam Axis
Rottary Drilling (wash Boring)

0.00M SP — Brownish colour Poorly graded fine to medium sand with fines. The fines are non plastic Pervious SP
1.00M SP — Brownish colour non plastic silty sand 100% Pervious SP
1.50M SP — Brownish colour non plastic silty sand 100% pervious SP
3.00M SP — Brownish colour Fine to medium decomposed rock sand with fines. The fines are non plastic Pervious SP
4.20M SP — Brownish colour Fine to medium sand with fines. The fines are non plastic Pervious SP.
6.00M

After 6.00 M, the water and sample completely lost.

Submitted by :- MR. K. Muthulingam S.T
MR. V. Balasundaram S.T.

S.T

Figure 3.8 – BH No. 05 of Vendrasan Dam

Figure 3.9 shows the orientation of this borehole 1 to borehole 4 which are located along the CH 470 m to CH 590 m. This cross sectional view clearly makes it visible that sand layers are present in the dam body as well as underneath the dam.

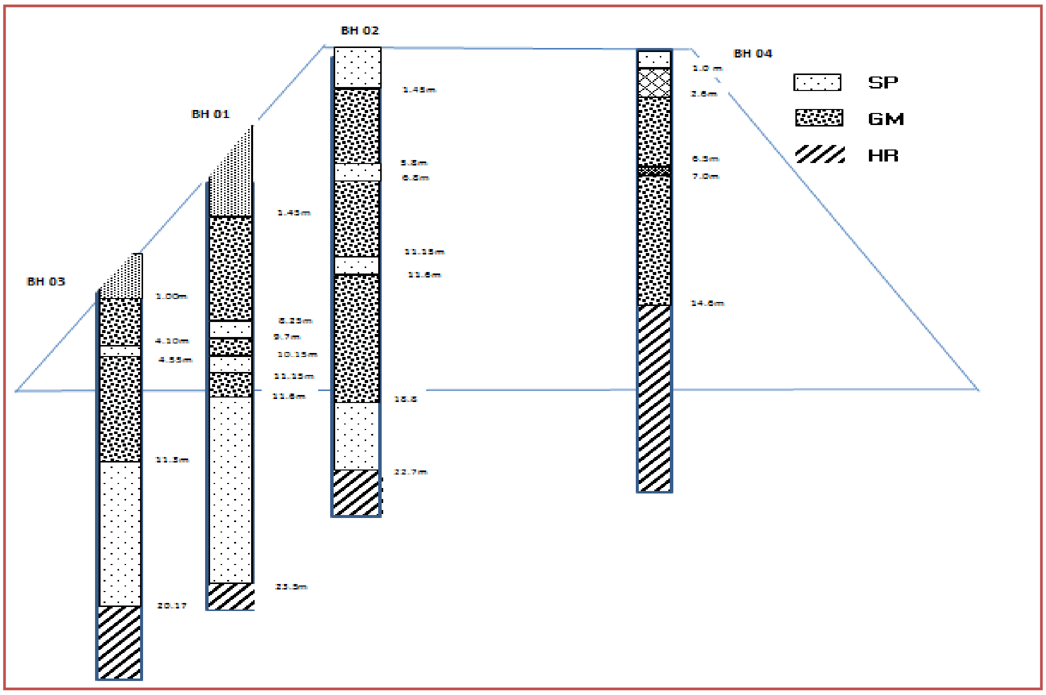


Figure 3.9 – sand layers in the dam body of Vendrasan Dam, CH 470- 590 m

3.1.2 Seepage Issue of the Vendrasan dam

From the above Figure 3.9 it can be seen that high permeability sand layers are present varying thickness through and beneath the dam body. It may lead the seepage path and create sink holes while making the downstream area boggy. Heavy vegetation could be observed downstream of the seepage section where high moisture available (Figure 3.10). Downstream improvements like loading berms and downstream filters may not clearly a permanently successful solution though applied temporary for this issue. Because seepages would appear again from a far location after sometimes. Grouting and sealing was also not succeeded twice, may be due to poor groutability of the material. So, investigate globally is timely to find out whether soil-cement-bentonite slurry cutoff walls are cost effective, durable, reliable and practical solution for mitigating seepages. Research objectives were defined so as to meet the requirements and detailed literature survey was conducted.

Figure 3.10– Dense vegetation along the toe of Vendrasan Dam

3.2 Planning and conducting laboratory testing

The series of laboratory testing were conducted to obtain the research objectives. To come out with a more suitable mix design of Soil-Cement-Bentonite (SCB) two soil composites were selected from the Vendrasan site, one is very close to the downstream and other is from little far borrow area. Moisture content, grain size distribution and atterburg limit tests were carried out for each composite for classification and designing purposes.

Considering the SCB walls in the literature and their design mixes and performances, four mix proportions were selected for testing as in Table 3.1. The prepared samples of each mix were cured for saturation and tested for 7 day, 14 day and 28 day saturated results. Saturated hydraulic conductivity is the paramount property of the material which was difficult to test with. Hence, Consolidation test with oedometer apparatus was utilized to overcome the difficulty. To attest the reliability of the permeability results derived from oedometer test, the mix proportions of mix no. 1 and 2 were selected so as tally with case study 2 presented in section 2.2.3.

Oedometer/consolidation tests were conducted for five loading increments, 25 kPa, 50 kPa, 100 kPa, 200 kPa, 300 kPa at each time steps, 7 day, 14 day and 28 day and thereby hydraulic conductivity (k) values were derived as described in section 3.2.2.. Unconsolidated undrained triaxial test (UU) was conducted to determine undrained cohesion (Cu) and thereby compressive strength (q_u) for 28 days saturated samples.

Proctor compaction test was conducted for composite 2 soil. Samples for consolidation and triaxial tests were prepared at maximum dry density (100% compaction) for soil of composite 2.

Table 3.1- Selected mix proportions for laboratory testing

Composite	**Mix No:**	**% Bentonite added by wt of soil**	**% Cement added by wt of soil**
Composite 1	1	3.3	8
	2	2.2	4
Composite 2	A	2	3
	B	1	3

3.2.1 Sample preparation procedure

The SCB backfill mixes for the research study were mixed according to the following procedure.

1.) Weigh the soil needed for each mix, pass it through a 12 mm (0.5”) sieve to remove large particles or clods, and set it aside for use in Step 5.

2.) Create 10% bentonite to water (by weight) slurry and set it aside to hydrate.

3.) Calculate the cement needed for each mix using the dosage rates in Table 3.1 to be applied to the soil quantity from Step 1.

4.) Mix the cement from Step 3 with water to create a one to one (by weight) cement to water grout.

5.) Add the cement-water grout to and mix it with the site soils from Step 1.

6.) Mix the bentonite slurry from Step 2 with the soil-grout mixture from step 5 to achieve a SCB backfill

7.) Continue to mix the SCB backfill until visually homogeneous.

8.) Cast the SCB mixture in cube moulds.

9.) Allow the SCB backfill specimens to cure prior to lab testing.

3.2.2 Laboratory testing procedure

Soil classification tests

Sieve analysis, liquid limit test, moisture content, proctor compaction tests were carried out for samples collected from vendrasan dam site (composite 1 & composite 2). Soil type identified and determined fine percentages (passing No. 200 sieve) for each composite. Optimum moisture content and maximum dry density values were calculated by proctor compaction test results.

Oedometer test

The samples were carefully taken to the oedometer ring and standard consolidation test was conducted by recording readings. Loading process was continued for 24 hours for each loading increment. Each sample was tested for five loading increments, 25kPa, 50kPa, 100kPa, 200kPa and 300kPa. Each mix was tested for three time steps, 7day, 14 day, and 28 day. Root time vs settlement graphs for each test were drawn with Taylor’s method. Coefficient of consolidation (C_v), coefficient of volume compressibility (m_v) and thereby hydraulic conductivity (k) were calculated for each consolidation graph using following equations.

$$m_v = \frac{(H_1-H_2)}{H_1} \times \frac{1}{(P_1-P_2)}$$

$$C_v = \frac{k}{(m_v \gamma_w)}$$

Where; m_v = Coefficient of volume compressibility

C_v = Coefficient of Consolidation

H_1 = Height of the specimen at the beginning of the stage in mm (ie. At the end of the previous stage)

H_2 = Height of the specimen at the end of that increment in mm

P_1 = Pressure applied to the specimen for the previous loading stage. (kPa)

P_2 = Pressure applied to the specimen for the loading stage being considered. (kPa)

K = Hydraulic Conductivity

γ_w = Unit weight of water

Triaxial test

Standard Unconsolidated Undrained (UU) triaxial test was carried out for 28 days old saturated samples prepared for each mix proportions of SCB. The test was conducted for cell pressures of 100kPa and 200kPa. Deviator stress vs axial strain graphs and shear stress vs normal stress graphs were drawn to determine undrained cohesion (Cu) values and thereby compressive strength (q_u) values for each samples.

3.3 Numerical Modeling and Analysis

Dam profiles along the critical section (CH 250-690 m) were modeled with available borehole data. A single borehole profile was assumed to be persisting throughout the critical section. Model 1 was defined assuming that profile of the borehole 1 is remaining unchanged throughout the dam section. Model 2 & 3 were also defined accordingly. A 1m thick Soil-Cement-Bentonite (SCB) wall was assumed to be implemented through the center of the dam at crest level up to the impermeable hard rock layer. Limit equilibrium analysis was conducted for stability analysis while finite element analysis for seepage analysis.

3.3.1 Finite Element modeling & Limit equilibrium analysis

SEEP/W software was used to conduct the finite element modeling for seepage analysis .Total head (H) boundary conditions and saturated material properties (hydraulic conductivity) were used for seep/w analysis. Seepage quantities (Flux) through the pre defined sections were determined before and after the application of the SCB cutoff wall. Flux values of minimum three sections were computed.

SLOPE/W software was used to conduct the limit equilibrium analysis for stability. The static stability of the downstream slope of the dam was also carried out at steady state condition to verify the applicability. Factor of safety values for the circular slip surfaces were determined by using Entry and Exit method. Adopted methods for the analysis are Ordinary, Bishop, Janbu and Morgentsern-Price.

Bishop method gives the factor of safety with respect to moment equilibrium (F_m), while the Janbu gives the factor of safety with respect to horizontal force equilibrium (F_f). Morgentsern-Price method considered both moment equilibrium (F_m) and horizontal force equilibrium (F_f). The general limit equilibrium formulation is based on two factor of safety equations and allows for a range of interslice shear-normal force assumptions. Figure 3.11 makes it possible to understand the differences between the factors of safety from the various methods, and to understand the influence of the selected interslice force function.

The interslice shear forces in the general limit equilibrium (GLE) method are handled with an equation proposed by Morgenstern and Price (1965).
The equation is:

$X = E \lambda f(x)$ (SLOPE/W theory book 2007)

where:

$f(x)$ = a function,
λ = the percentage (in decimal form) of the function used,
E = the interslice normal force, and
X = the interslice shear force.

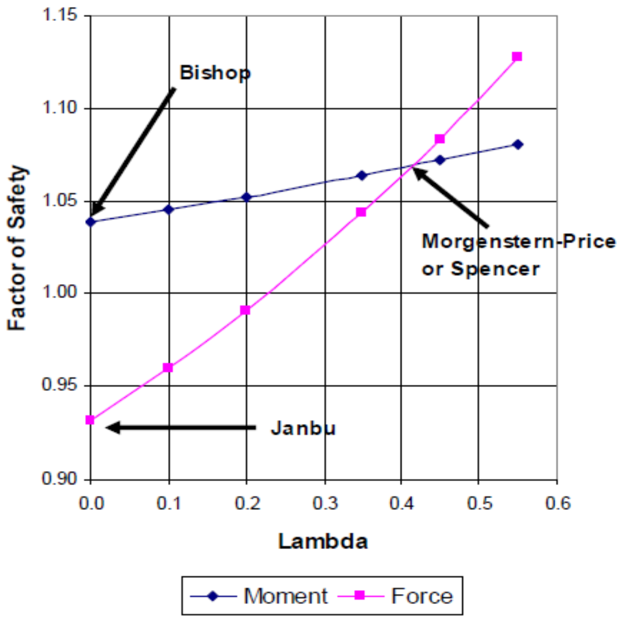


Figure 3.11- Factor of Safety versus lambda (λ) plot

3.3.2. Material Properties

General values of soil properties were adopted in modeling the subsurface strata for the soils which no values of soil properties were found by the literature of the Vendrasan Dam. Cohesion (c'), Angle of Friction (Ø'), Saturated Density (γ_{sat}) and Hydraulic Conductivity (k) values as shown in Table 3.2 were required for SEEP/W and SLOPE/W analysis. For SCB cutoff wall the laboratory results were adopted with, the most suitable mix proportion's results.

Table 3.2 – Soil properties adopted for numerical analysis

Soil Type	**C (kPa)**	**Ø (^{0})**	**γ_{sat} (kN/m^3)**	**k (m/sec)**
SC	3	35	22	10^{-7}
GM	0	34	21	10^{-6}
SM	2	33	22	10^{-5}
SP	0	30	19.5	10^{-4}
CL	10	28	16	10^{-8}
SC/SM	3	32	22	10^{-6}
SP/SM	2	30	20	10^{-5}
SCB	49	0	22	10^{-10}

3.4 Data analysis and Interpretation

3.4.1 Laboratory test results

Basic soil classification test results are presented in Table 3.3 and data sheets are annexed.

Composite 1 classified as SM material consists with more fine percentage than composite 2 which is also classified as SM material.

Table 3.3 – Soil classification results of two composites of Vendrasan dam

Composite ID	Soil Group	Moisture Content (%)	*Grain size (% passing)*			Fines Content (%)
			# 4	*# 30*	*# 200*	
Composite 1	Silty Sand (SM)	9.25	100	41.6	41.6	41.6
Composite 2	Silty Sand (SM)	8.13	100	30.5	30.5	30.5

Oedometer test results

The calculated Coefficient of Consolidation (Cv), Coefficient of Volume Compressibility (m_v) and Hydraulic Conductivity (k) values for each time steps are presented in following tables separately for each mix proportions as below. The data sheets and graphs are attached to the annexure.

Table 3.4 - Oedometer test results for Mix 01

Table 3.5 - Oedometer test results for Mix 02

Table 3.6 - Oedometer test results for Mix A

Table 3.7 - Oedometer test results for Mix B

Table 3.4 – Oedometer test results for Mix 01

MIX NO	Days	Load (kPa)	Cv (m^2/yr)	m_v (m^2/kN)	k (m/sec)
1	7 days	25	22.7402	0.000296	$2.09387 *10^{-9}$
		50	13.5536	0.000149	$6.28208 *10^{-10}$
		100	10.8813	0.000182	$6.1605*10^{-10}$
		200	21.9825	0.0000793	$5.425*10^{-10}$
		300	9.6156	0.0000615	$1.8399*10^{-10}$
	14 days	25	8.4255	0.000908	$2.3798 *10^{-9}$
		50	6.4425	0.000089023	$1.7841 *10^{-10}$
		100	3.74765	0.00009027	$1.05239*10^{-10}$
		200	3.709	0.00004894	$5.6465*10^{-11}$
		300	4.4148	0.00004253	$5.8416*10^{-11}$
	28 days	25	4.95232	0.00032	$4.9297 *10^{-10}$
		50	3.2557	0.000106	$1.07353 *10^{-10}$
		100	4.844	0.000082912	$1.2493*10^{-10}$
		200	10.8021	0.000051797	$1.74053*10^{-10}$
		300	8.8333	0.00003267	$8.9779*10^{-11}$

Table 3.5 – Oedometer test results for Mix 02

MIX NO	Days	Load (kPa)	Cv (m^2/yr)	m_v (m^2/kN)	k (m/sec)
2	7 days	25	9.2088	0.000376	$1.07709 *10^{-9}$
		50	10.9342	0.000153	$5.21909 *10^{-10}$
		100	13.3685	0.000124	$5.1465*10^{-10}$
		200	13.19013	0.0000914	$3.75027*10^{-10}$
		300	21.39188	0.0000577	$3.84213*10^{-10}$
	14 days	25	8.2578	0.00041818	$1.0742*10^{-9}$
		50	6.3193	0.00018386	$3.67427 *10^{-10}$
		100	11.696	0.000163362	$5.9436*10^{-10}$
		200	11.4628	0.000076799	$2.73848*10^{-10}$
		300	12.5672	0.00004603	$1.79964*10^{-10}$
	28 days	25	6.1139	0.000144	$2.7387*10^{-10}$
		50	7.0744	0.000156626	$3.4468 *10^{-10}$
		100	4.5594	0.000080686	$1.1443*10^{-10}$
		200	3.9875	0.000056735	$7.03739*10^{-11}$
		300	4.7659	0.00004791	$7.01286*10^{-11}$

Table 3.6 – Oedometer test results for Mix A

MIX NO	Days	Load (kPa)	Cv (m^2/yr)	m_v (m^2/kN)	k (m/sec)
A	7 days	25	7.738	0.000218	$5.2474*10^{-10}$
		50	6.5125	0.0001489	$3.01683*10^{-10}$
		100	7.5779	0.0002546	$6.002757*10^{-10}$
		200	6.7987	0.0001782	$3.7688*10^{-10}$
		300	11.3383	0.000152353	$5.3735*10^{-10}$
	14 days	25	5.6851	0.000188	$3.3247*10^{-10}$
		50	6.05299	0.000225126	$4.2389*10^{-10}$
		100	10.8912	0.00016487	$5.5858*10^{-10}$
		200	7.4239	0.00016028	$3.7016*10^{-10}$
		300	10.3377	0.000015573	$5.00796*10^{-10}$
	28 days	25	10.1068	0.000108	$3.3955*10^{-10}$
		50	7.687	0.0002428	$5.8059*10^{-10}$
		100	1.2137	0.000504	$1.9028*10^{-10}$
		200	8.00045	0.00020165	$4.2476*10^{-10}$
		300	7.04543	0.00012157	$2.66433*10^{-10}$

Table 3.7 – Oedometer test results for Mix B

MIX NO	Days	Load (kPa)	Cv (m^2/yr)	m_v (m^2/kN)	k (m/sec)
B	7 days	25	4.9523	0.000124	$1.91025*10^{-10}$
		50	11.0726	0.000126	$4.3536*10^{-10}$
		100	12.1816	0.00012282	$4.65*10^{-10}$
		200	7.53	0.000101368	$2.3744*10^{-10}$
		300	9.627	0.00006711	$2.00976*10^{-10}$
	14 days	25	8.2578	0.0001555	$3.9959*10^{-10}$
		50	6.9305	0.000103464	$2.23057*10^{-10}$
		100	7.3258	0.0001161	$2.6466*10^{-10}$
		200	10.4194	0.00008273	$2.6814*10^{-10}$
		300	7.74069	0.00005738	$1.3817*10^{-10}$
	28 days	25	4.95232	0.000052	$8.01077*10^{-11}$
		50	5.6691	0.00017625	$3.10811*10^{-10}$
		100	4.7572	0.00004693	$6.9454*10^{-11}$
		200	5.4321	0.000103836	$1.7546*10^{-10}$
		300	4.9516	9.82826E-05	$1.513857*10^{-10}$

Graphs drawn on Oedometer test results

Series of graphs were drawn on calculated data. Coefficient of consolidation (Cv) variation with load increments for all four (4) mix proportions were plotted for 7 day, 14 day and 28 day samples separately as listed below.

Figure 3.12 - Cv vs Load for 7 day old sample

Figure 3.13 - Cv vs Load for 14 day old sample

Figure 3.14 - Cv vs Load for 28 day old sample

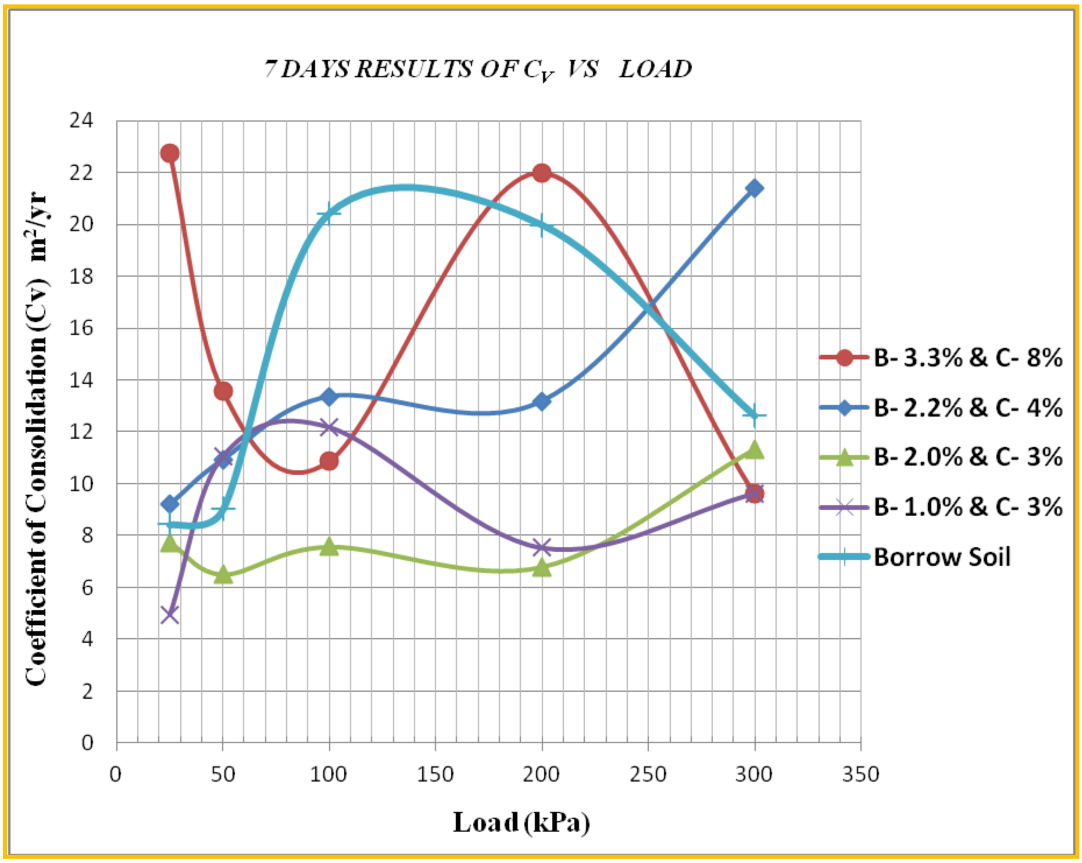


Figure 3.12 – Cv vs Load for 7 days old sample

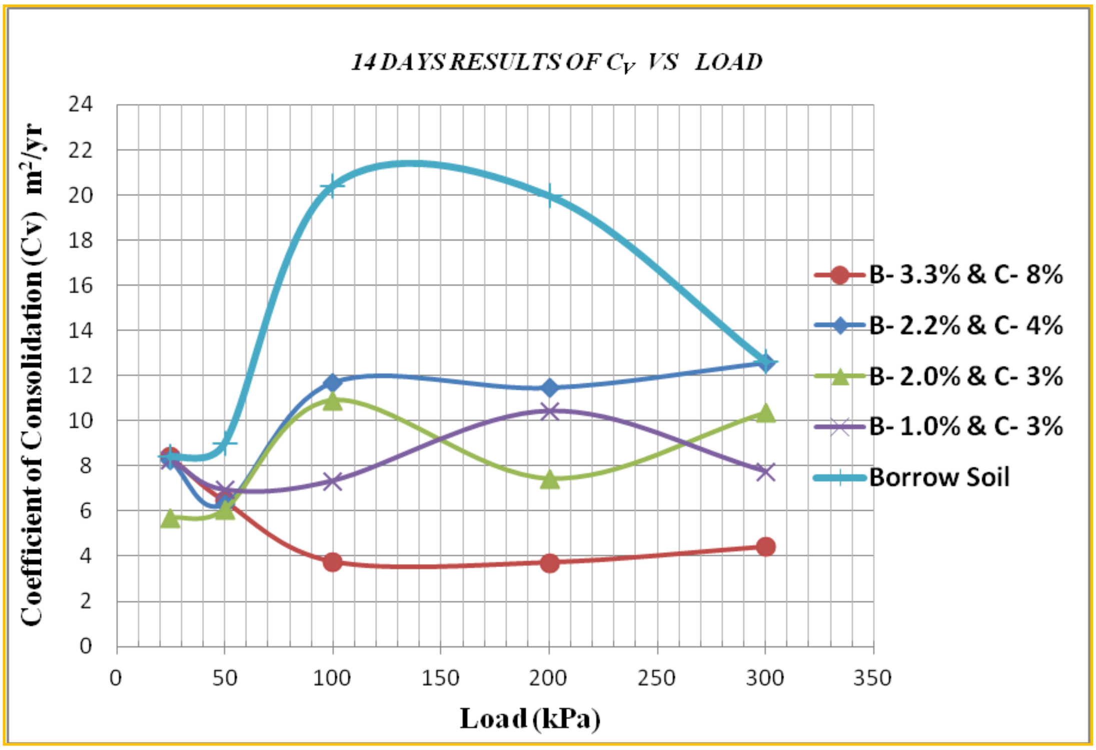


Figure 3.13 – Cv vs Load for 14 days old

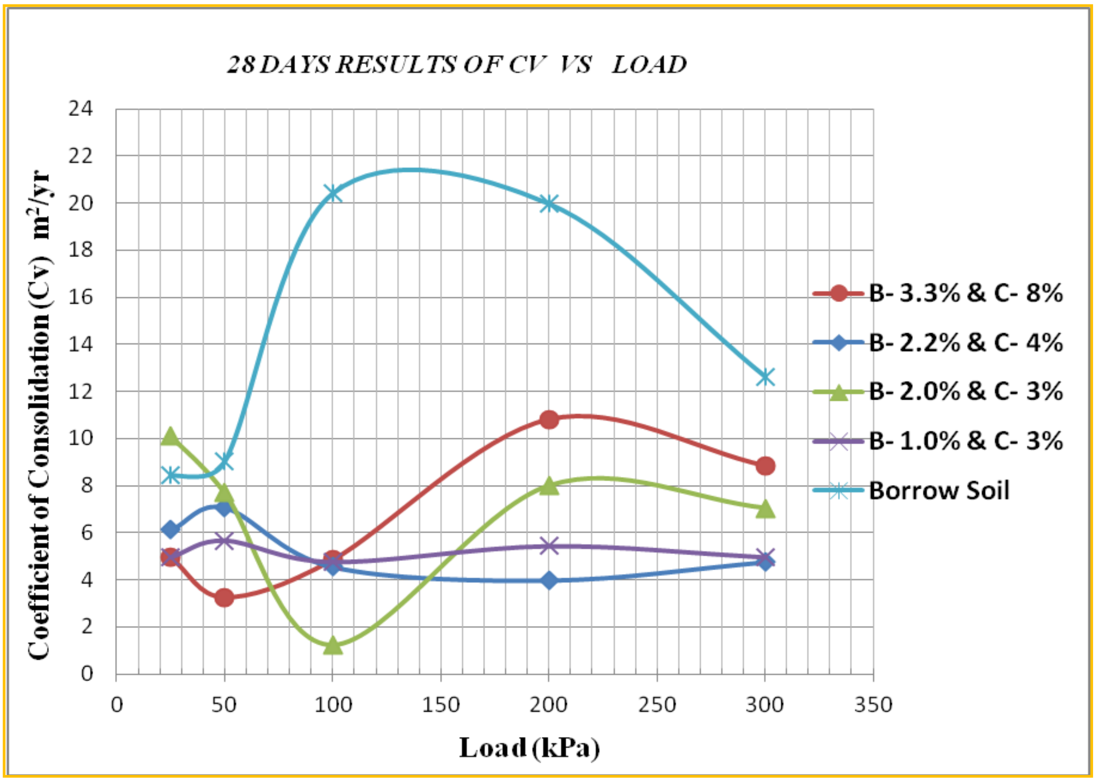


Figure 3.14 – Cv vs Load for 28 days old

Coefficient of Consolidation (Cv) values of all four mixes are comparatively reducing with time and lower than the borrow soil.

Coefficient of volume compressibility (m_v) variation with load increments for all four (4) mix proportions were plotted for 7 day, 14 day and 28 day samples separately as listed below.

Figure 3.15 - m_v vs Load for 7 days old sample

Figure 3.16 - m_v vs Load for 14 days old sample

Figure 3.17 - m_v vs Load for 28 days old sample

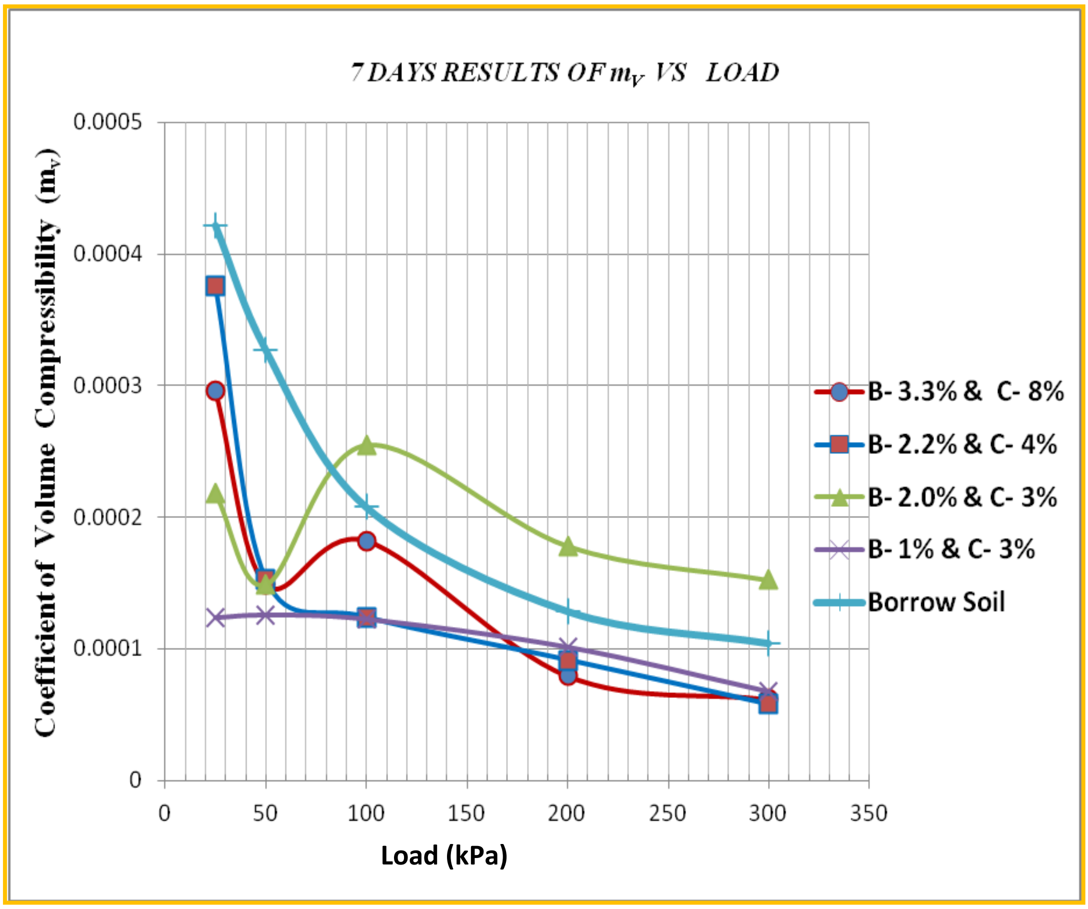


Figure 3.15 – m_v vs Load for 7 day old sample

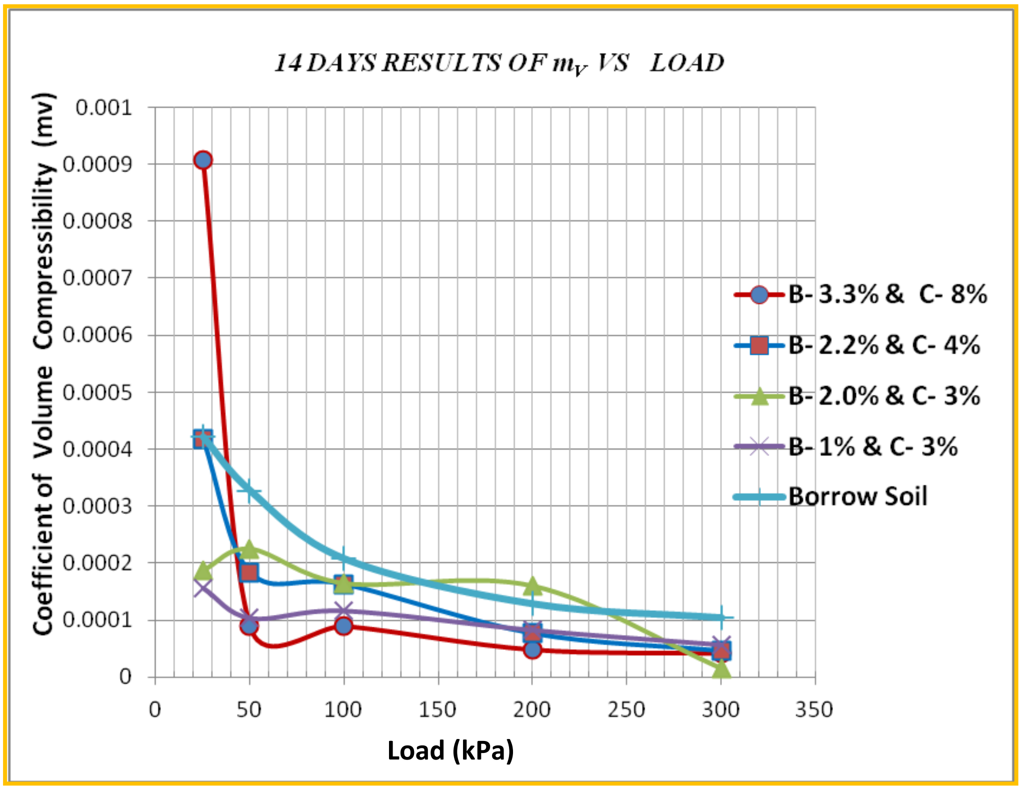


Figure 3.16 – m_v vs Load for 14 day old sample

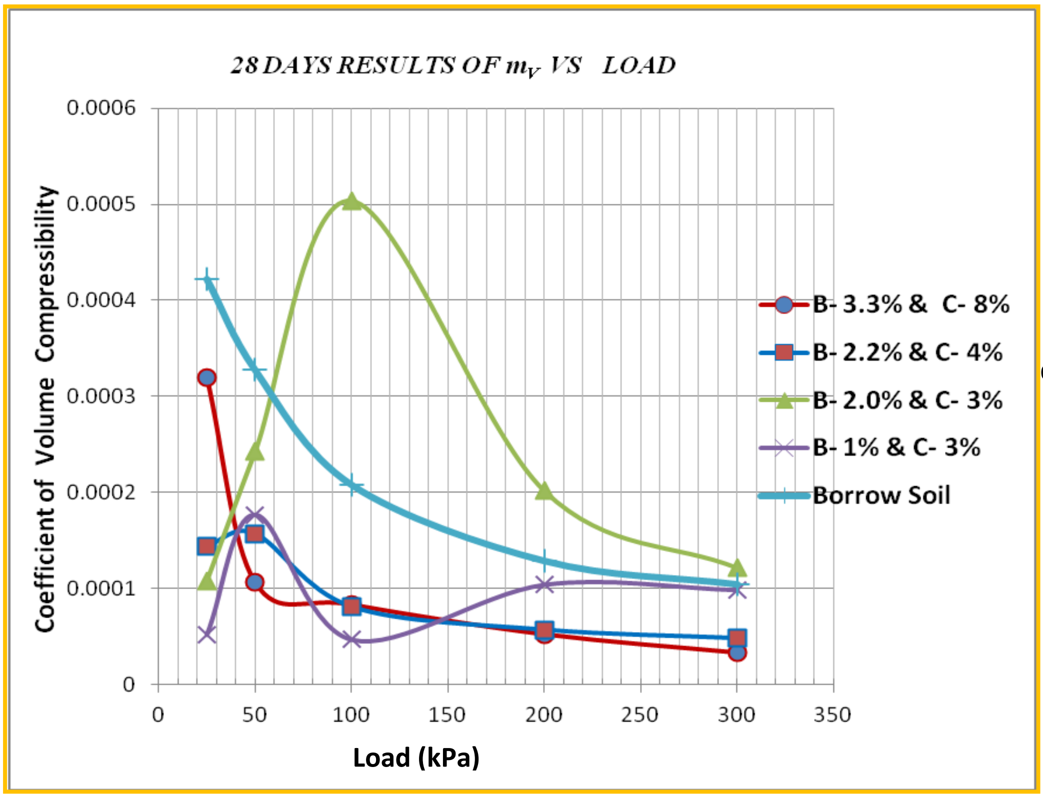


Figure 3.17 – m_v vs Load for 28 day old sample

Coefficient of volume compressibility (m_v) values of all four mixes are comparatively reducing with time and lower than the borrow soil.

Hydraulic conductivity (k) variation with load increments for all four (4) mix proportions were plotted for 7 day, 14 day and 28 day samples separately as listed below.

Figure 3.18 - k vs Load for 7 day old sample

Figure 3.19 - k vs Load for 14 day old sample

Figure 3.20 - k vs Load for 28 day old sample

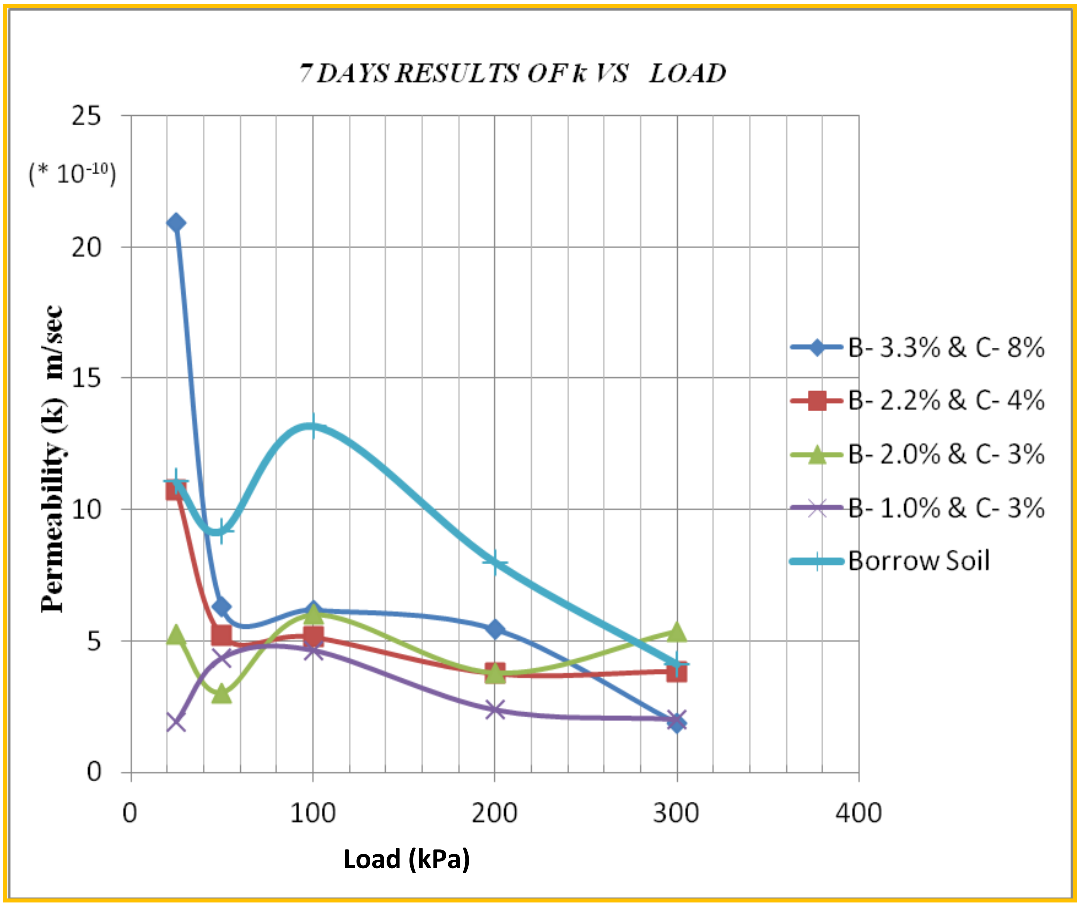


Figure 3.18 – k vs. Load for 7 day old sample

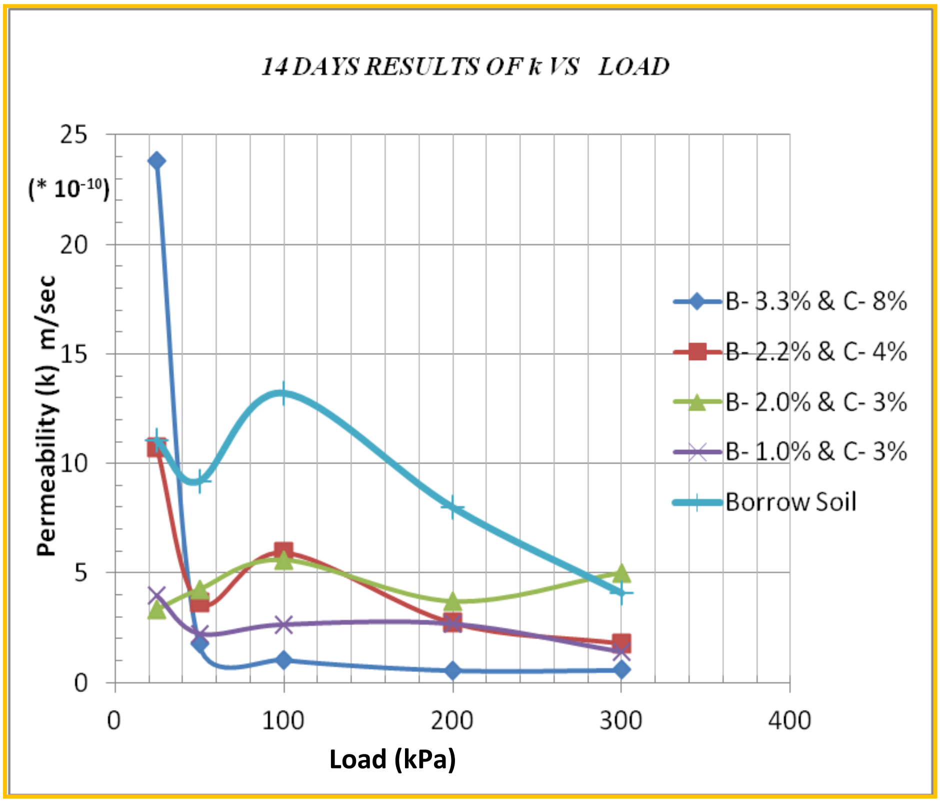


Figure 3.19 – k vs Load for 14 day old sample

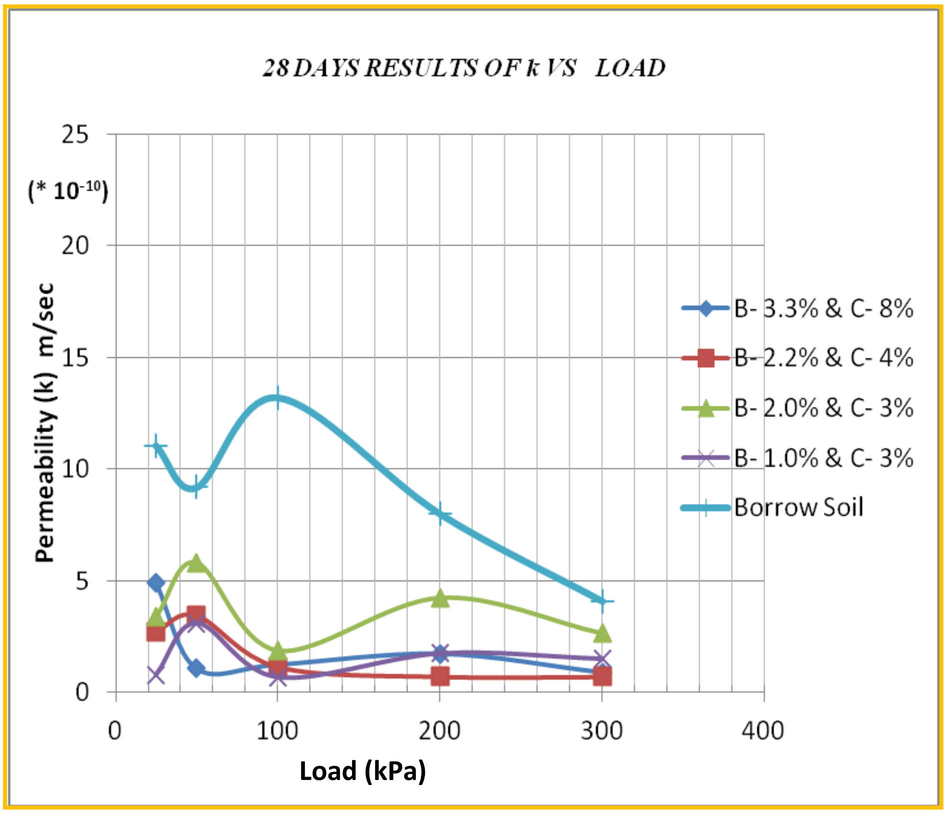


Figure 3.20 – k vs Load for 28 day old sample

Hydraulic conductivity (k) values of all four mixes are comparatively reducing with time and lower than the borrow soil. Further, k decrease with the loading increments.

The Coefficient of Consolidation (Cv), Coefficient of Volume Compressibility (m_v) and Hydraulic Conductivity (k) values variation with load and time for each saturated mix proportions are plotted for analyzing purposes. Untreated borrow sample (Composite 2) data were also plotted in the same graph for ease of comparison.

The Coefficient of Consolidation (Cv) vs load at time steps for each mixes are shown in following Figures.

Figure 3.21 - Cv vs Load with time for Mix No. 01 (**B - 3.3% & C - 8%)**

Figure 3.22 - Cv vs Load with time for Mix No. 02 (**B – 2.2% & C - 4%)**

Figure 3.23 - Cv vs Load with time for Mix No. A (**B – 2.0% & C - 3%)**

Figure 3.24 - Cv vs Load with time for Mix No. B (**B – 1.0% & C - 3%)**

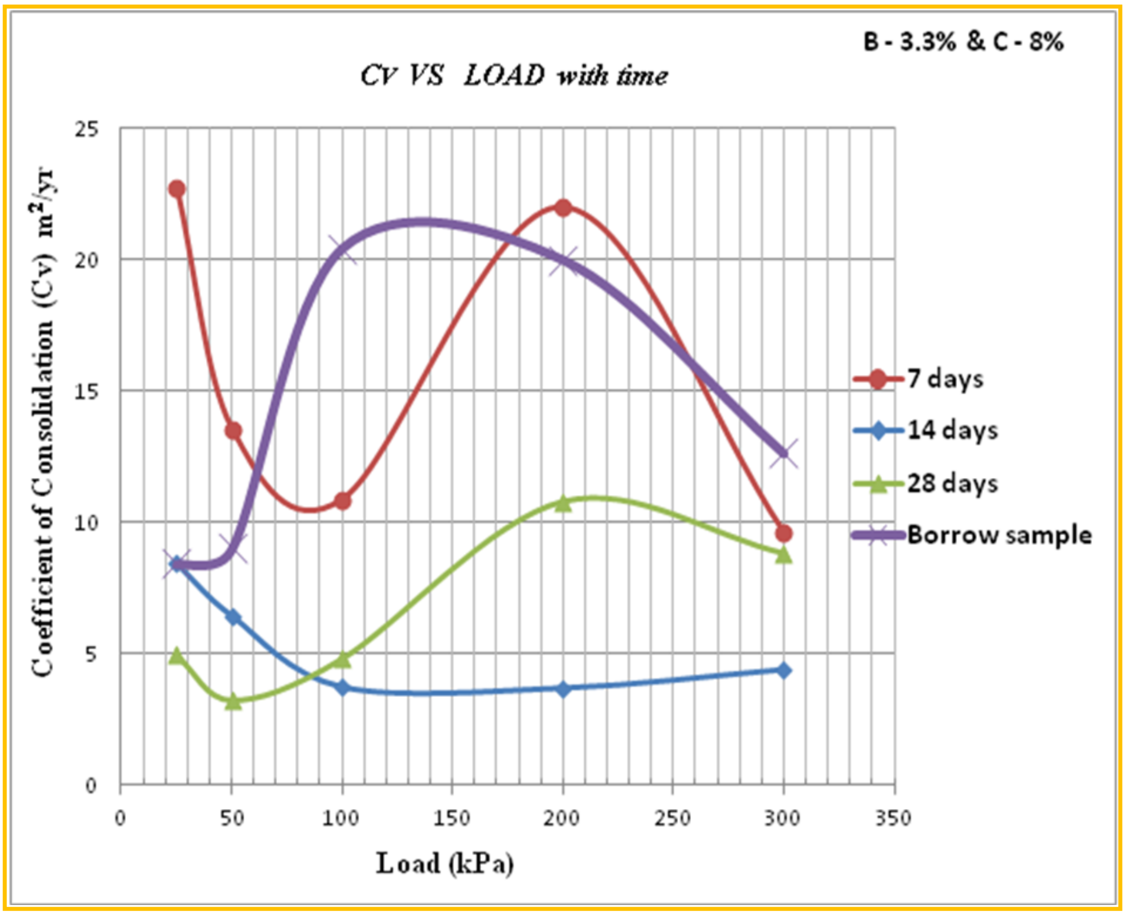


Figure 3.21 – Cv vs Load with time for Mix No. 01

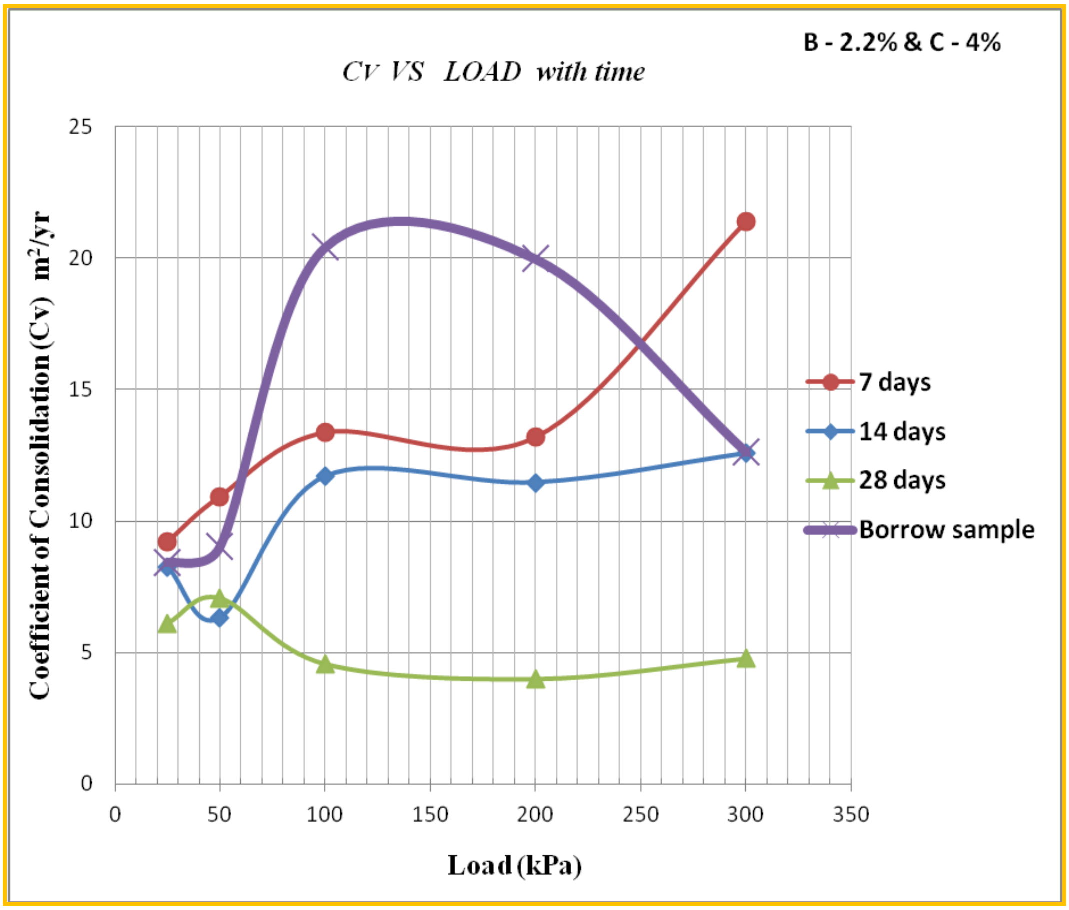


Figure 3.22 – Cv vs Load with time for Mix No. 02

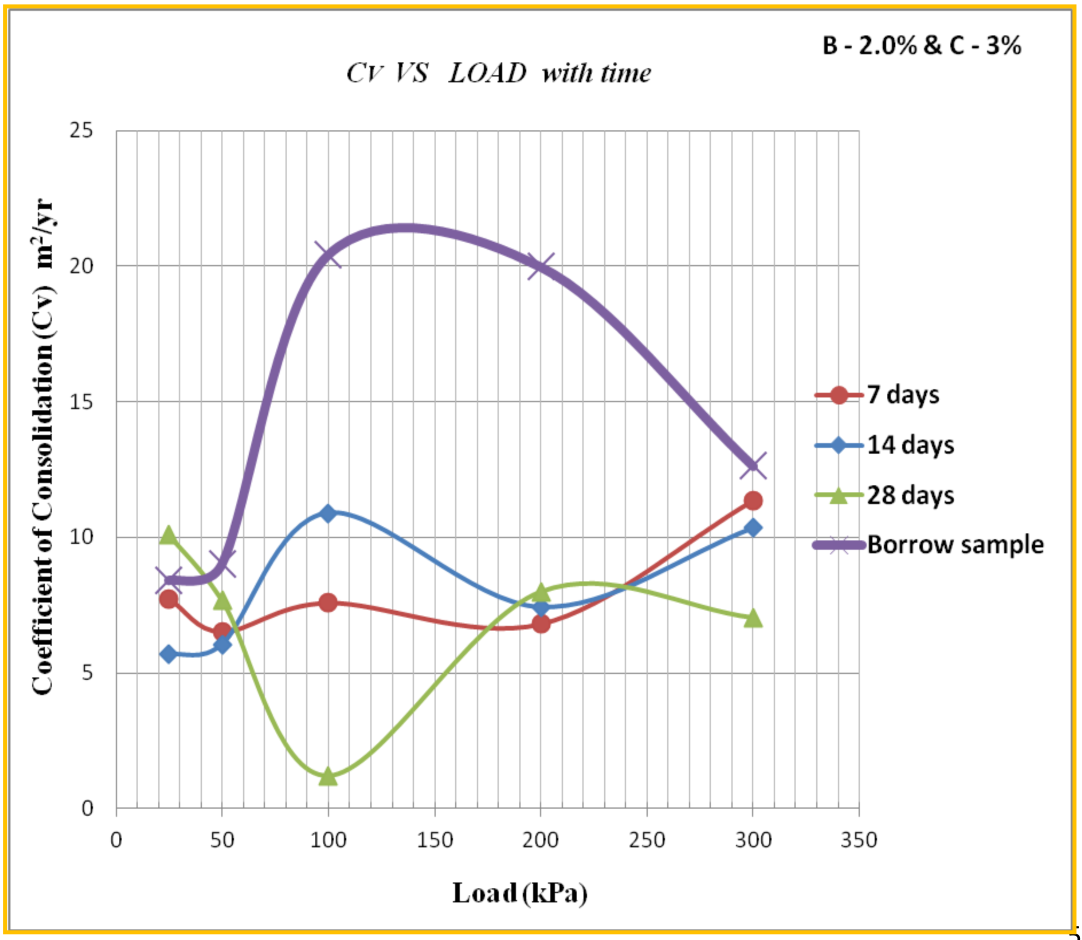


Figure 3.23 – Cv vs Load with time for Mix No. A

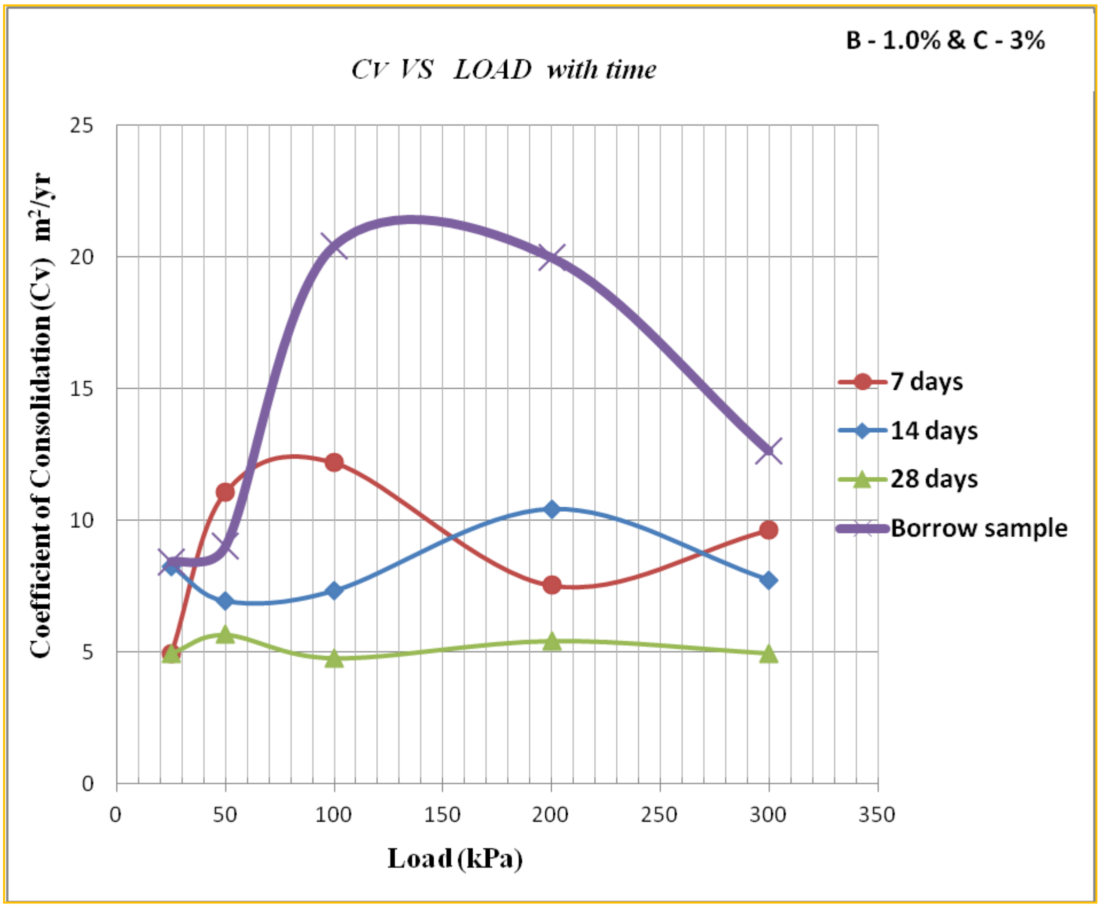


Figure 3.24 – Cv vs Load with time for Mix No. B

Coefficient of Consolidation values (Cv) are generally decrease with time and clearly lower than the values of borrow sample.

The Coefficient of Volume Compressibility (m_v) vs load at time steps for each mixes are shown in following Figures.

Figure 3.25 - m_v vs Load with time for Mix No. 01 (**B - 3.3% & C - 8%)**

Figure 3.26 - m_v vs Load with time for Mix No. 02 (**B – 2.2% & C - 4%)**

Figure 3.27 - m_v vs Load with time for Mix No. A (**B – 2.0% & C - 3%)**

Figure 3.28 - m_v vs Load with time for Mix No. B (**B – 1.0% & C - 3%)**

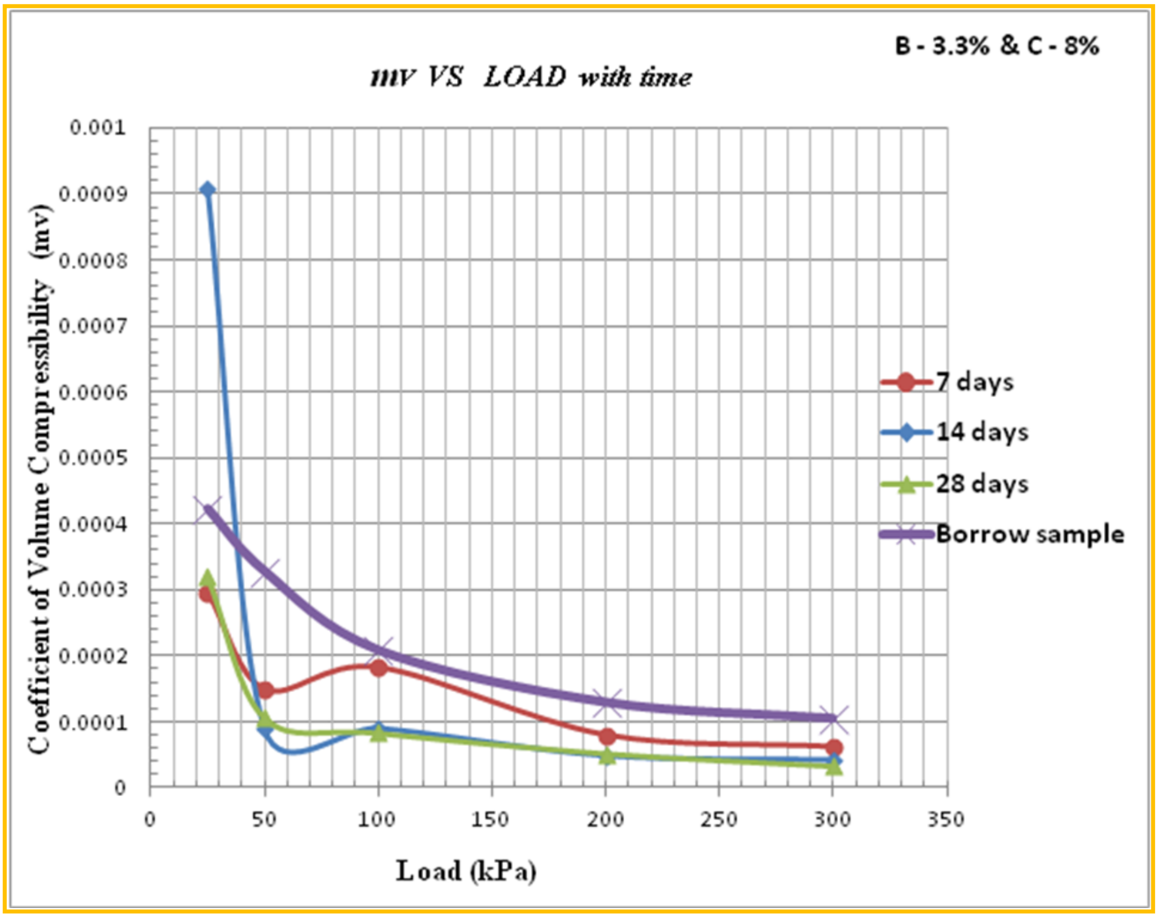


Figure 3.25 – m_v vs Load with time for Mix No.01

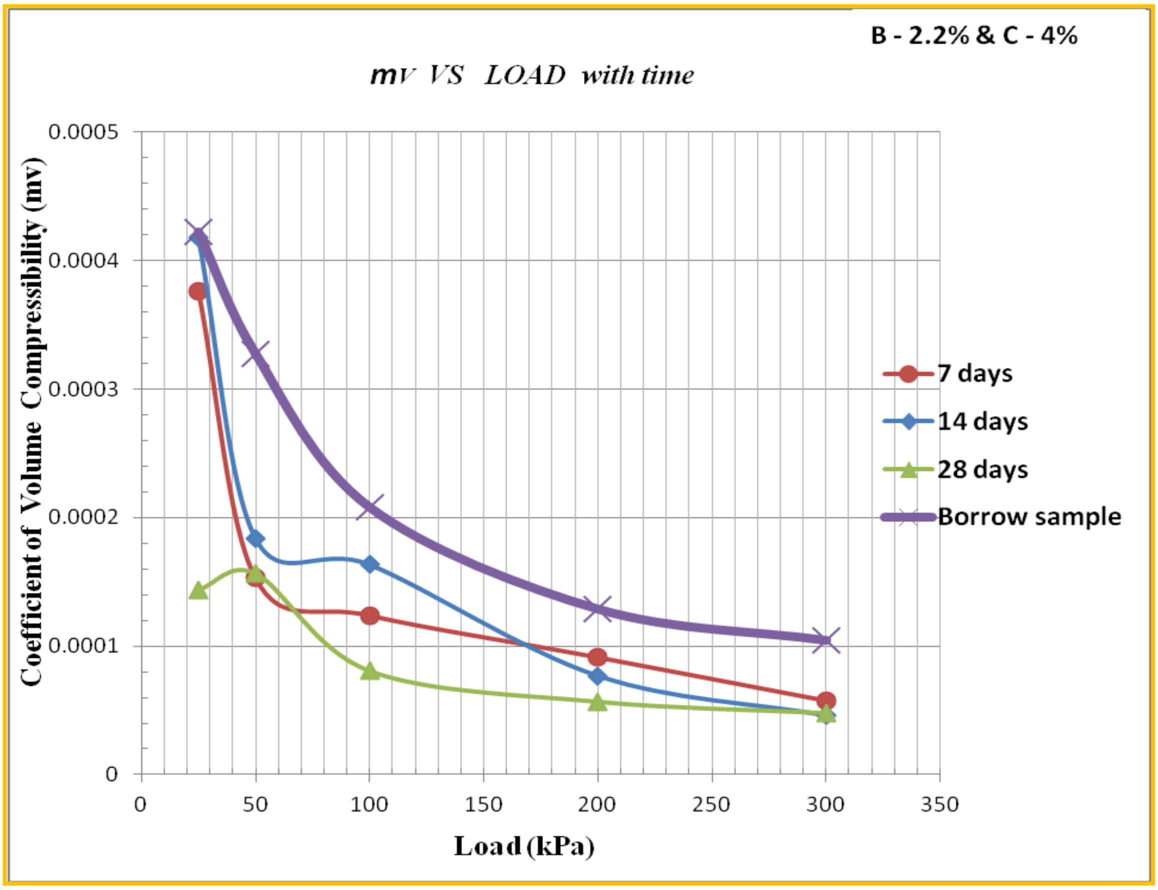


Figure 3.26 – m_v vs Load with time for Mix No.02

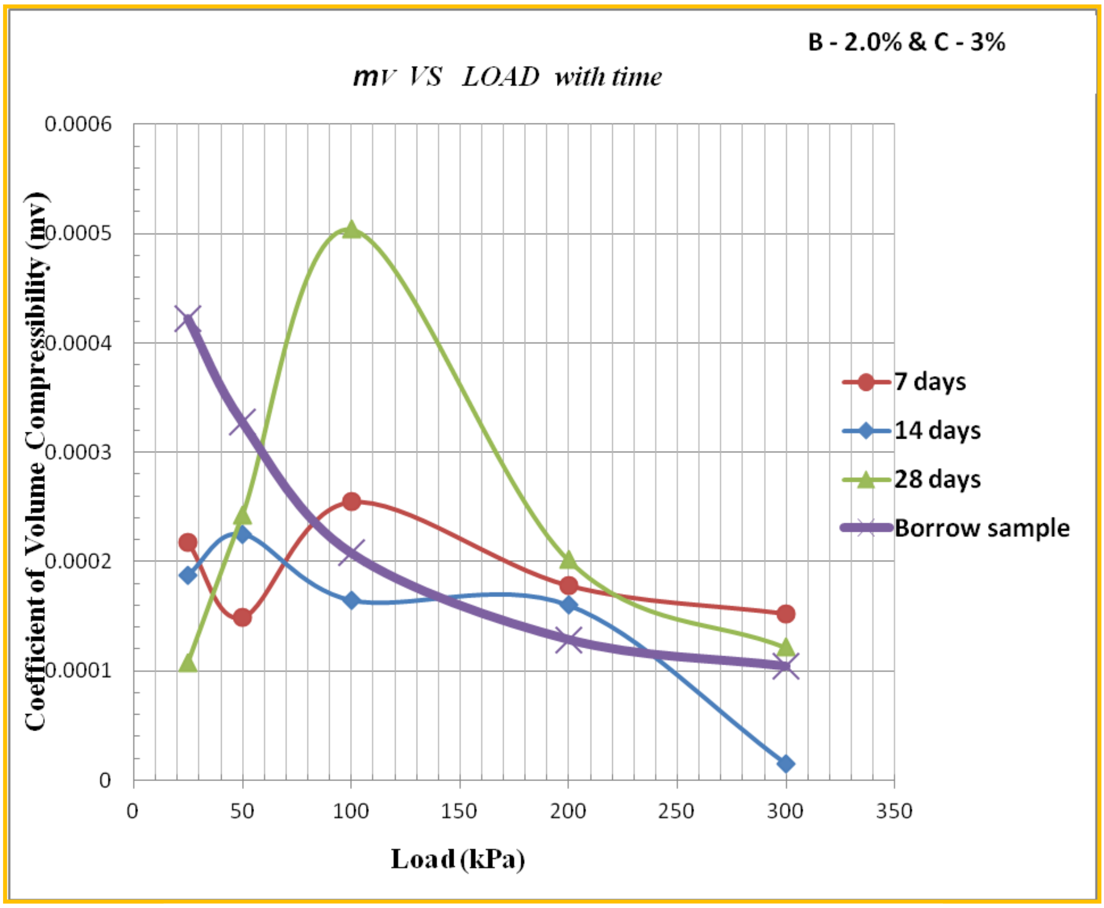


Figure 3.27 – m_v vs Load with time for Mix No. A

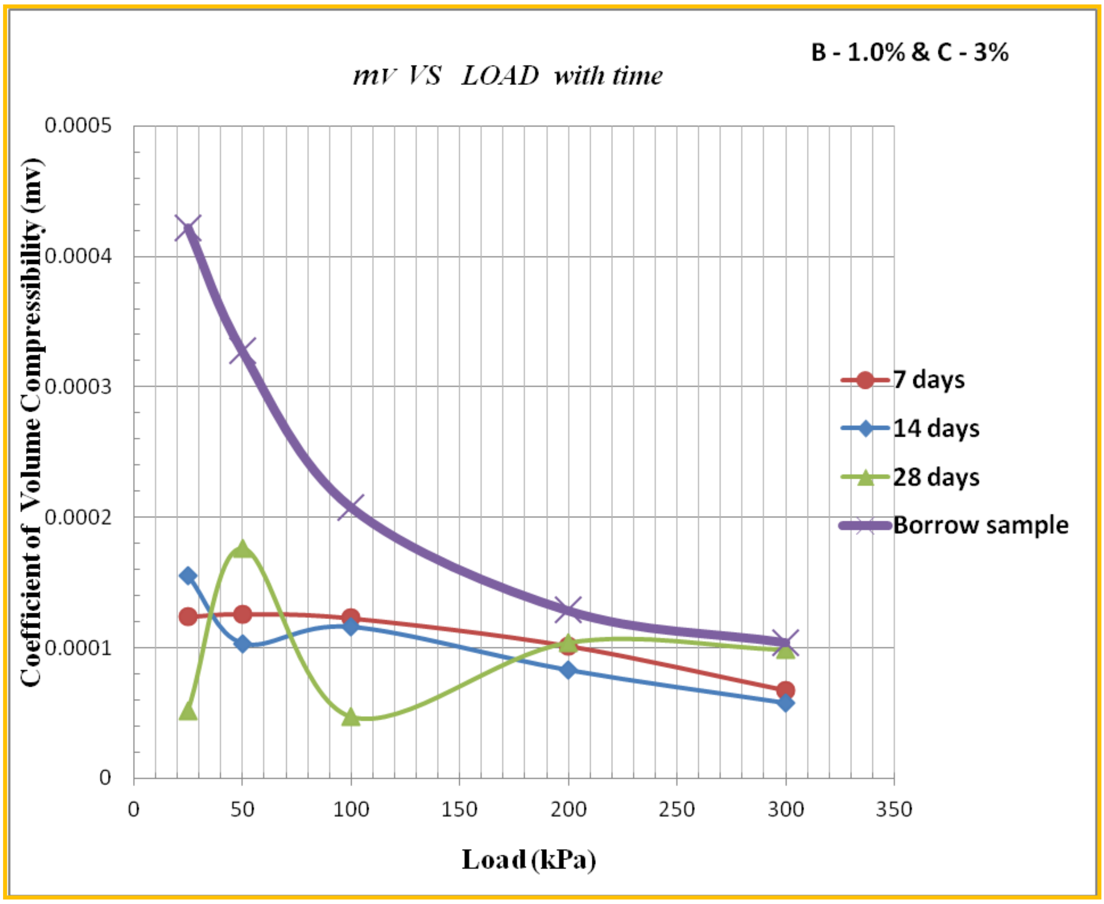


Figure 3.28 – m_v vs Load with time for Mix No. B

Coefficient of volume compressilbility values (m_v) are generally decrease with time and clearly lower than the values of borrow sample.

The Hydraulic Conductivity (k) vs Load at time steps for each mixes are shown in following Figures.

Figure 3.29 - k vs Load with time for Mix No. 01 (**B - 3.3% & C - 8%)**

Figure 3.30 - k vs Load with time for Mix No. 02 (**B – 2.2% & C - 4%)**

Figure 3.31 - k vs Load with time for Mix No. A (**B – 2.0% & C - 3%)**

Figure 3.32 - k vs Load with time for Mix No. B (**B – 1.0% & C - 3%)**

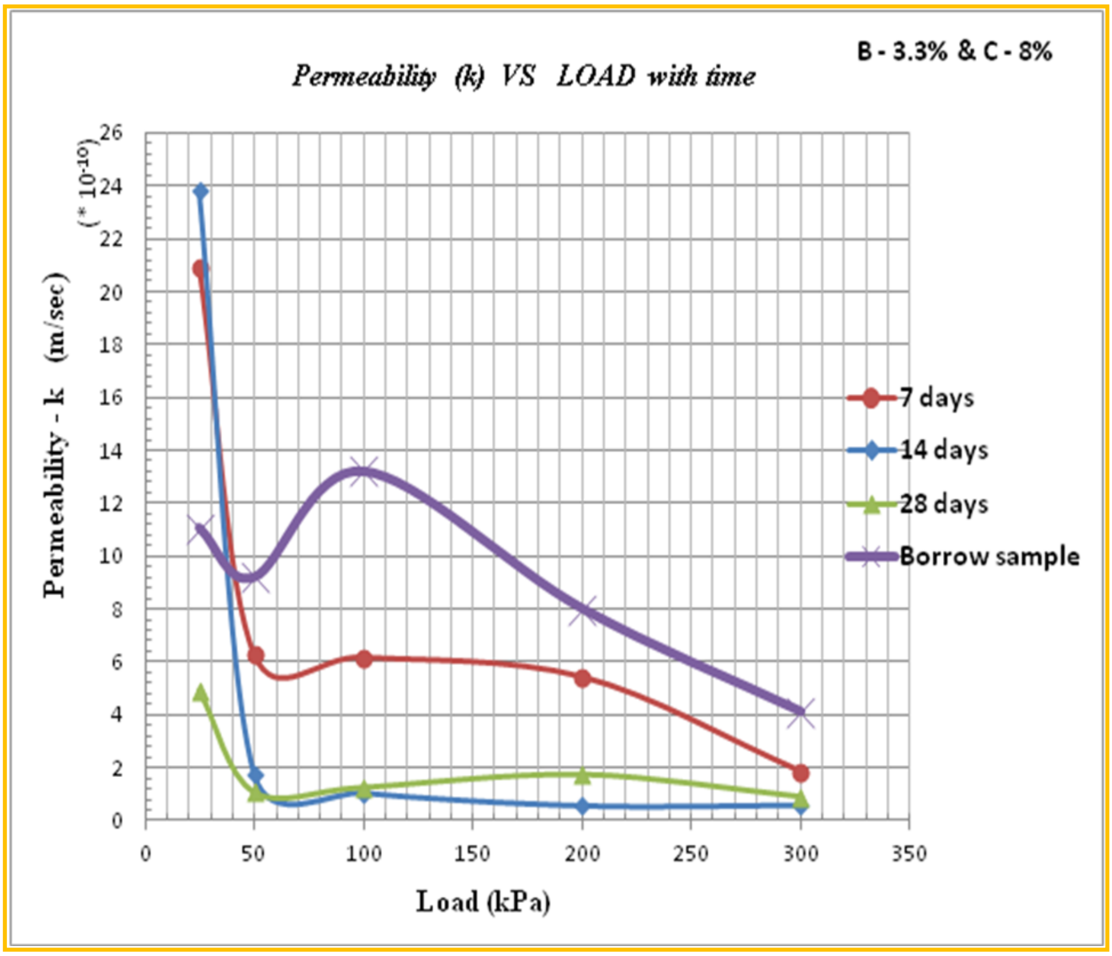


Figure 3.29 – k vs Load with time for Mix No.01

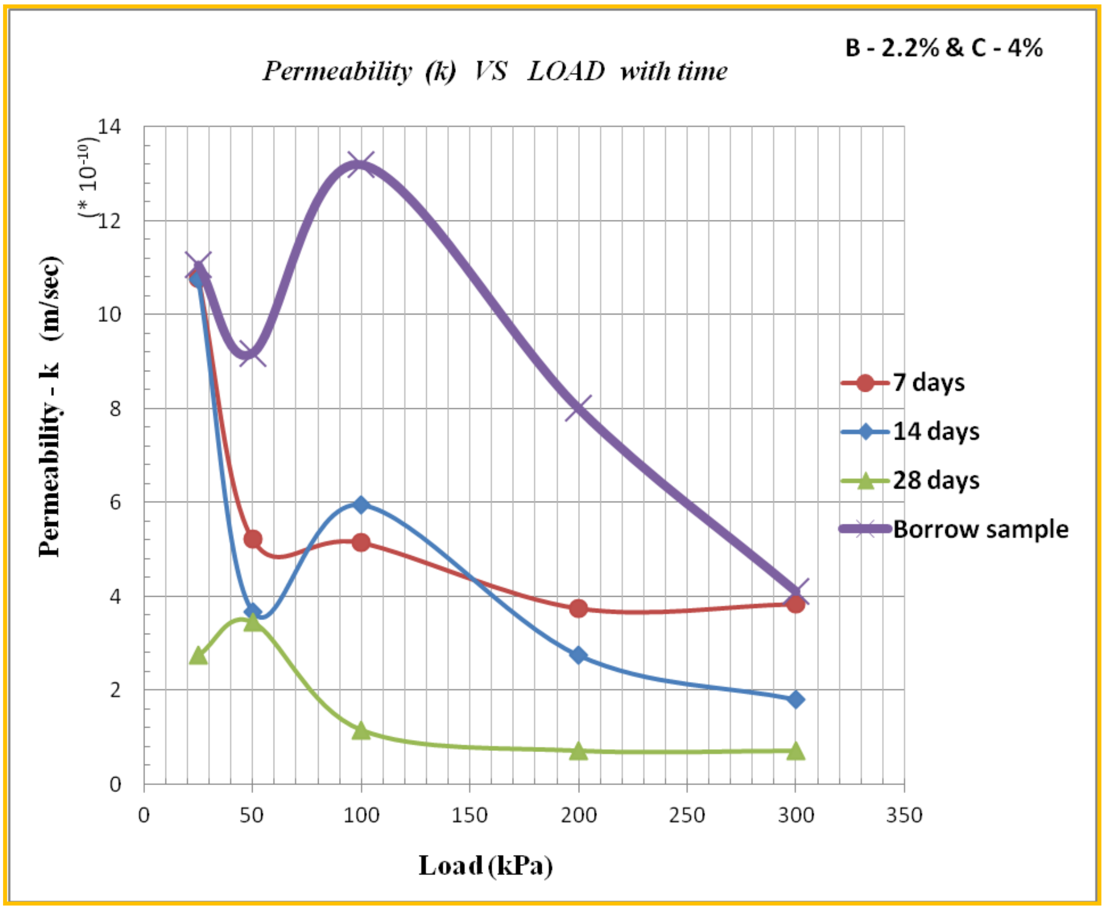


Figure 3.30 – k vs Load with time for Mix No.02

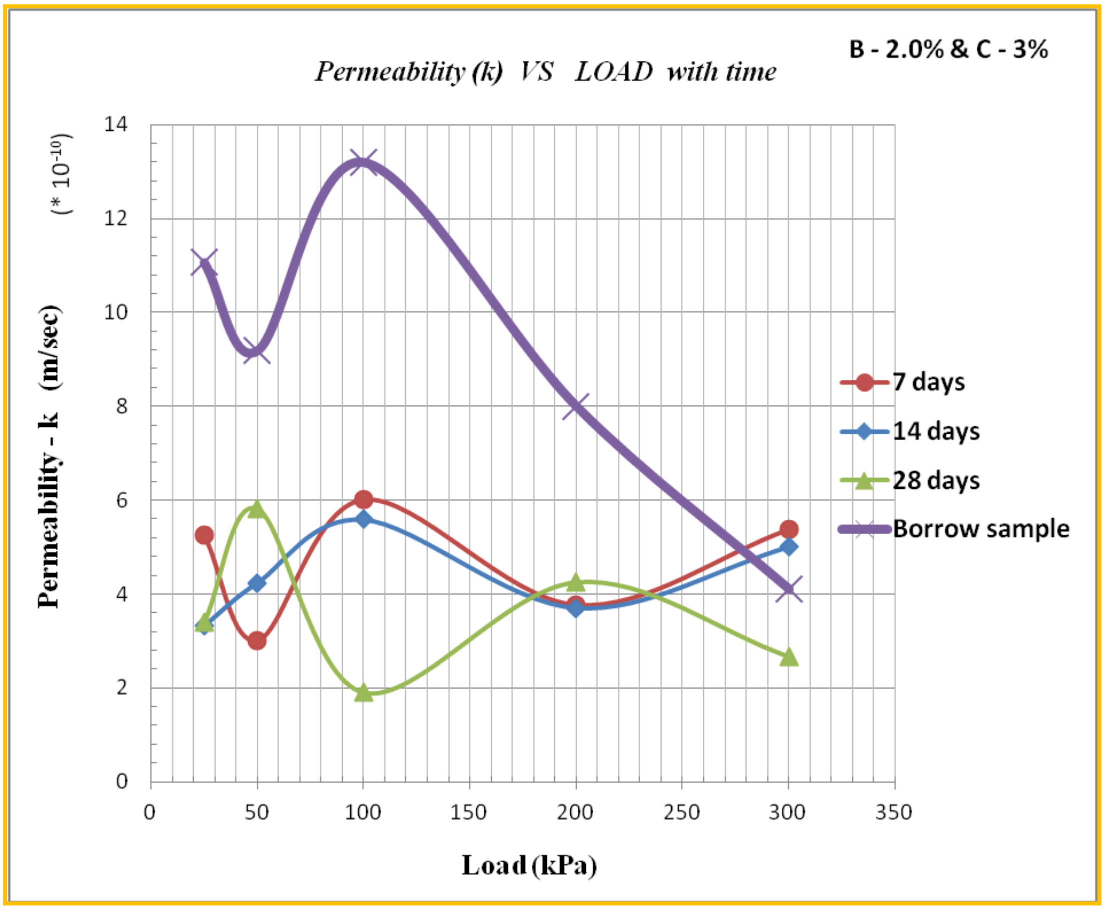


Figure 3.31 – k vs Load with time for Mix No. A

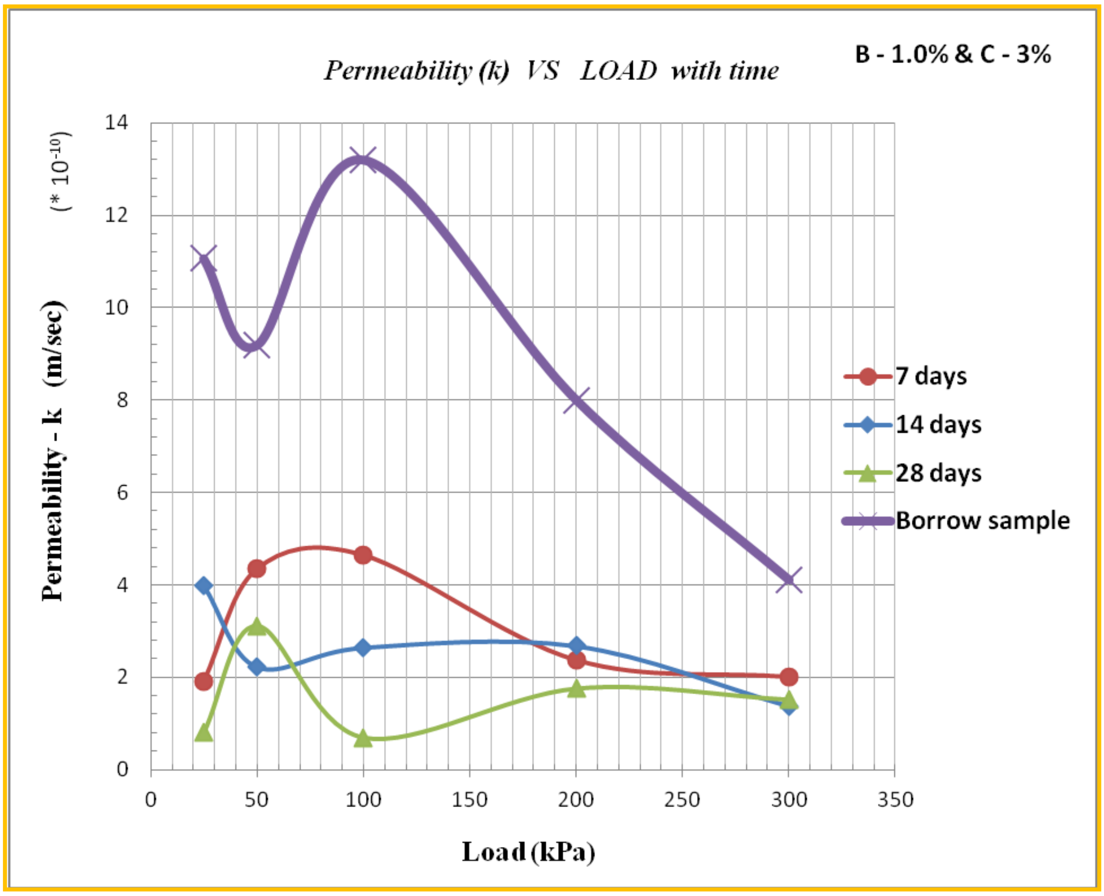


Figure 3.32 – k vs Load with time for Mix No. B

These graphs shows that the hydraulic Conductivity values (k) are generally decreasing with time and clearly lower than the values of borrow sample. Permeability values of Mix No B are comparatively lower than the other mixes. Further, there's a trend of decreasing of k with increasing of load.

When compare the mix no.1 and 2 from same composite (composite 1), mix no.2 exhibits lower permeability values.

When compare the mix no. A and B, which are from same composite (composite 2) and same cement percentage, mix no. B exhibits lower permeability which has lower bentonite percentage than mix no. A

Triaxial test results

Triaxial test data and plotted graphs are presented as follows.

Mix No. 01 Results are as follows;

Figure 3.33 - Deviator Stress Vs Axial Strain for mix No.01

Figure 3.34 - Shear Stress Vs Normal Stress for mix No.01

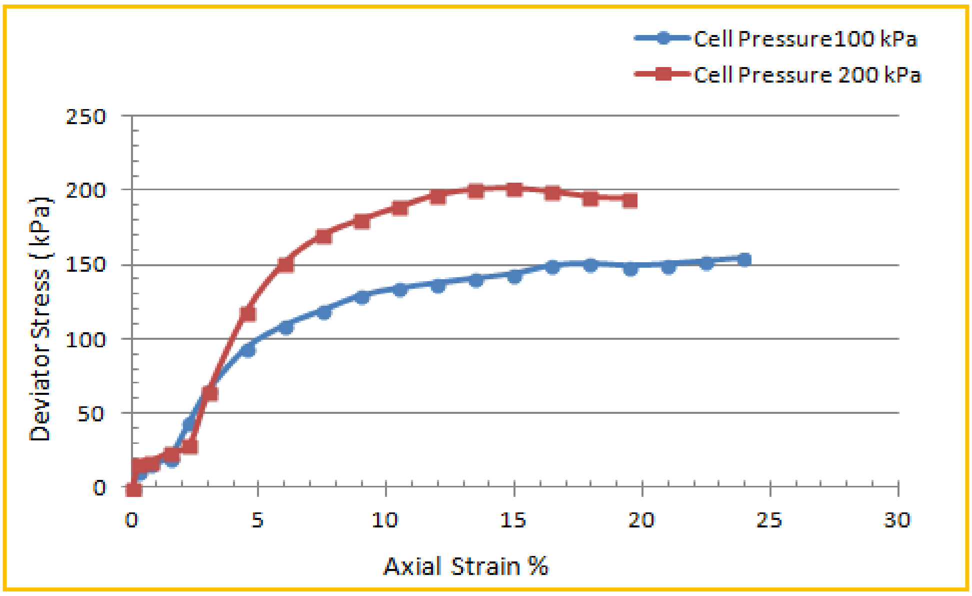


Figure 3.33 – Deviator Stress Vs Axial Strain for mix No.01

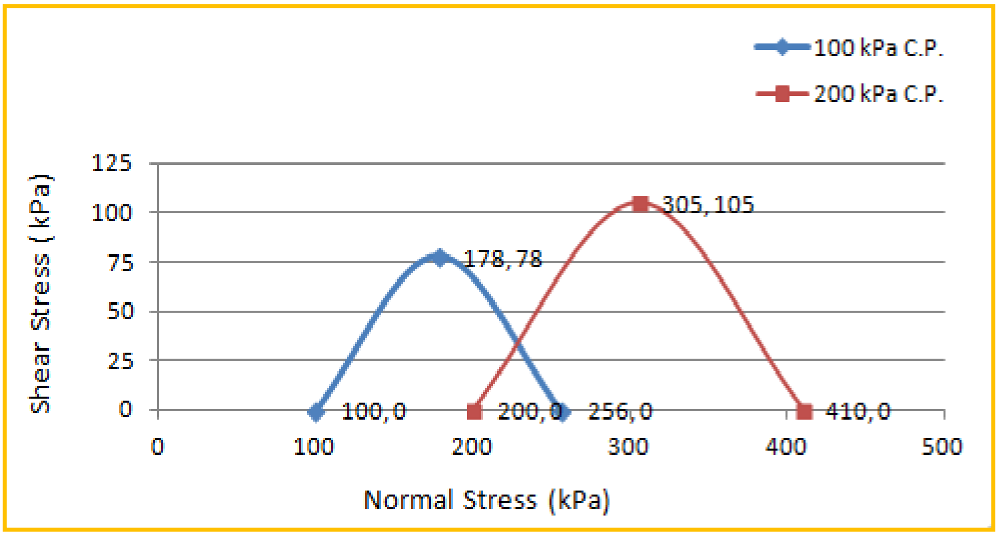


Figure 3.34 – Shear Stress Vs Normal Stress for mix No.01

Unconsolidated Undrained Cohesion (C_u) = (78+105)/2 = 91.5 kPa

Unconsolidated Undrained Shear Strength (q_u) = 183 kPa

Mix No. 02, Results are as follows;

Figure 3.35 - Deviator Stress Vs Axial Strain for mix No.02

Figure 3.36 - Shear Stress Vs Normal Stress for mix No.02

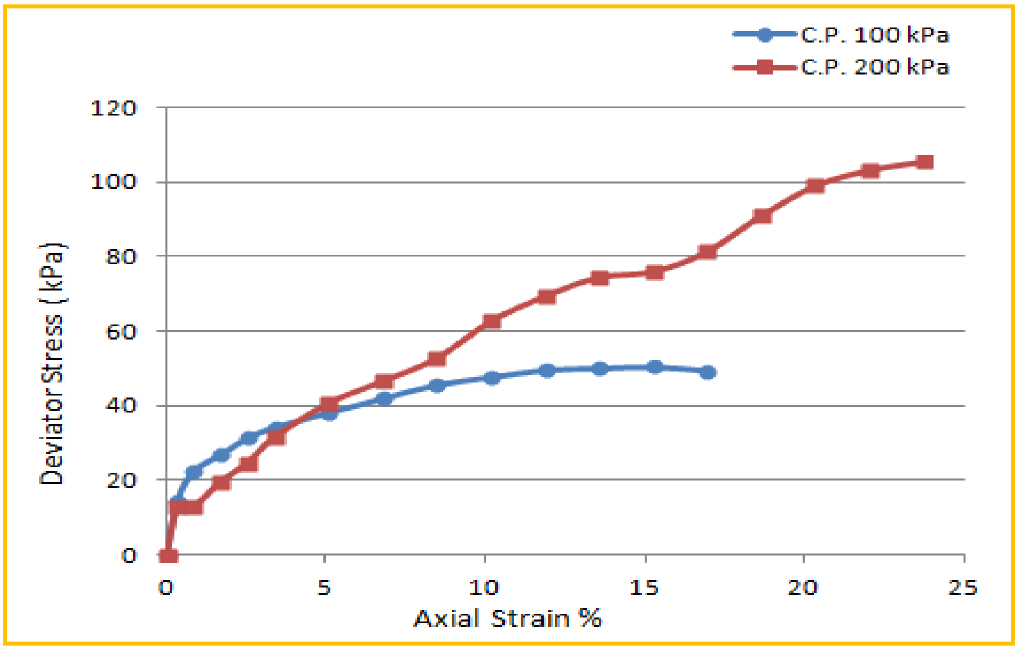


Figure 3.35 – Deviator Stress Vs Axial Strain for mix No.02

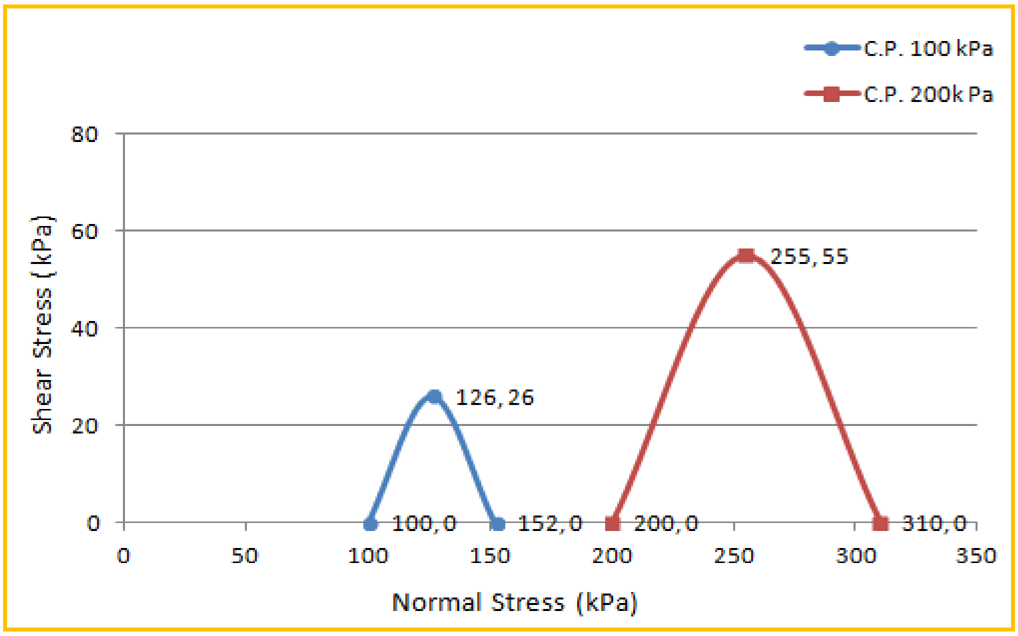


Figure 3.36 – Shear Stress Vs Normal Stress for mix No.02

Unconsolidated Undrained Cohesion (C_u) = (26+55)/2 = 40.5 kPa

Unconsolidated Undrained Shear Strength (q_u) = 81 kPa

Mix No. A, Results are as follows;

Figure 3.37 - Deviator Stress Vs Axial Strain for mix No.A

Figure 3.38 - Shear Stress Vs Normal Stress for mix No.A

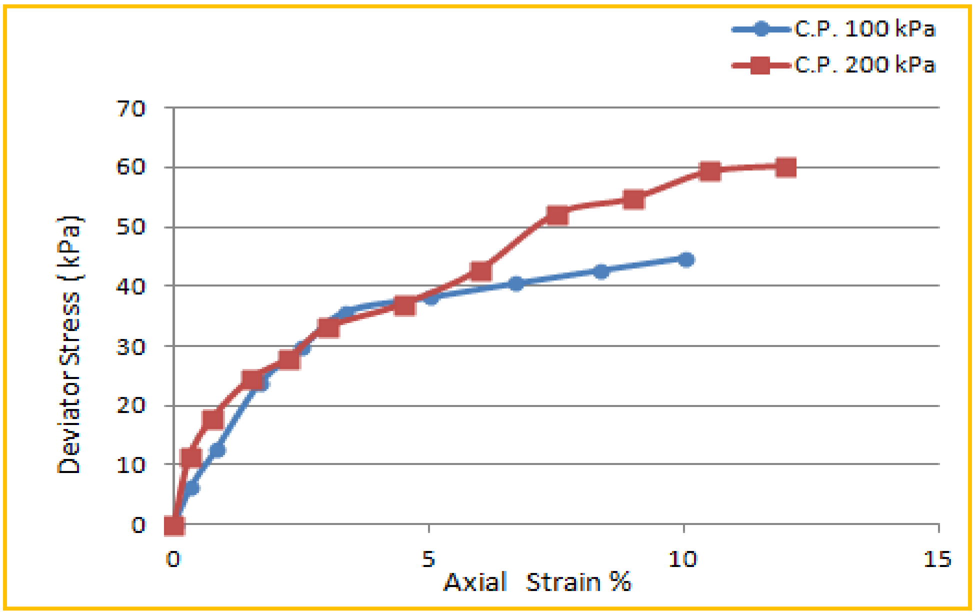


Figure 3.37 – Deviator Stress Vs Axial Strain for mix No.A

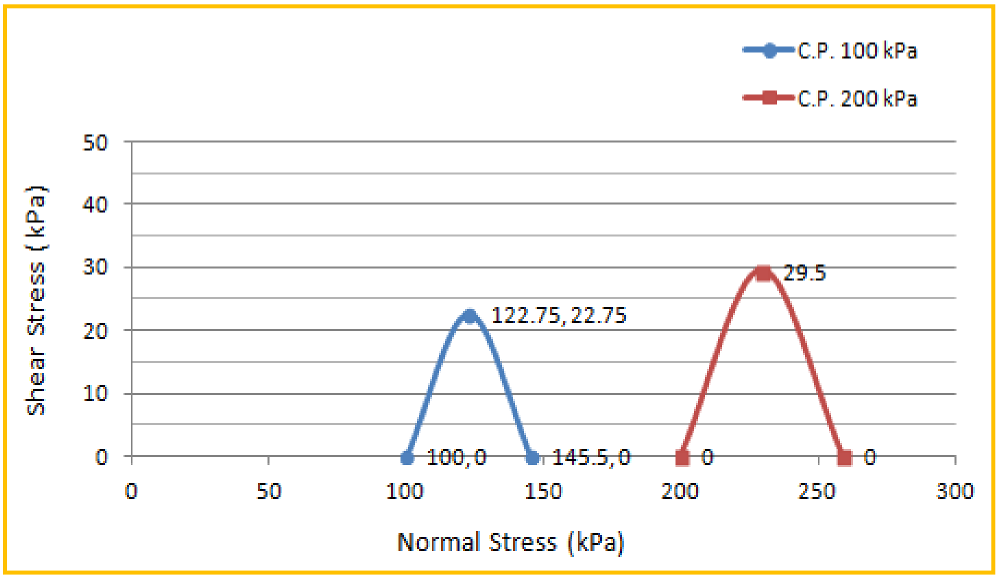


Figure 3.38 – Shear Stress Vs Normal Stress for mix

Unconsolidated Undrained Cohesion (C_u) = (22.75+29.5)/2 = 26.125 kPa

Unconsolidated Undrained Shear Strength (q_u) = 52.25 kPa

Mix No. B, Results are as follows;

Figure 3.39 - Deviator Stress Vs Axial Strain for mix No.B

Figure 3.40 - Shear Stress Vs Normal Stress for mix No.B

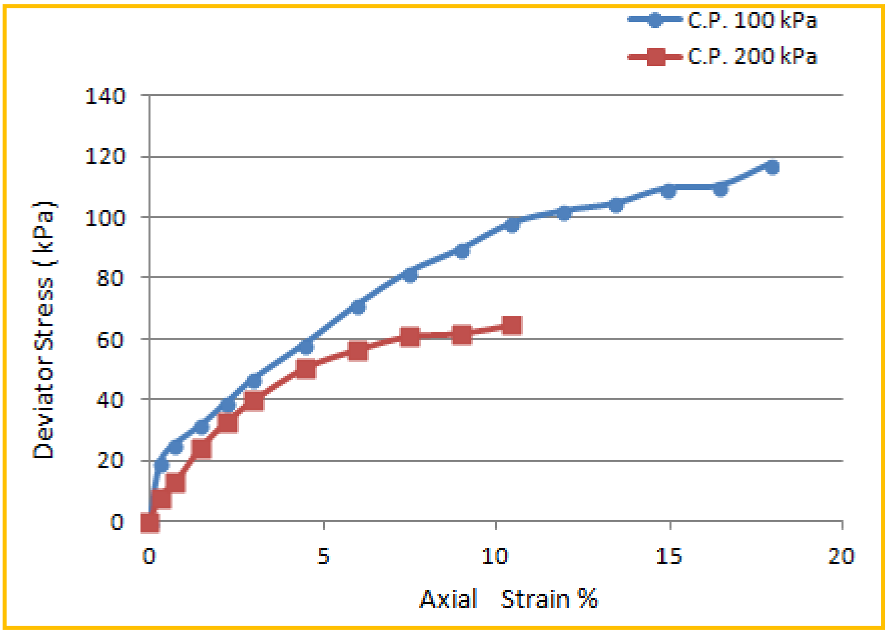


Figure 3.39 – Deviator Stress Vs Axial Strain for mix No.B

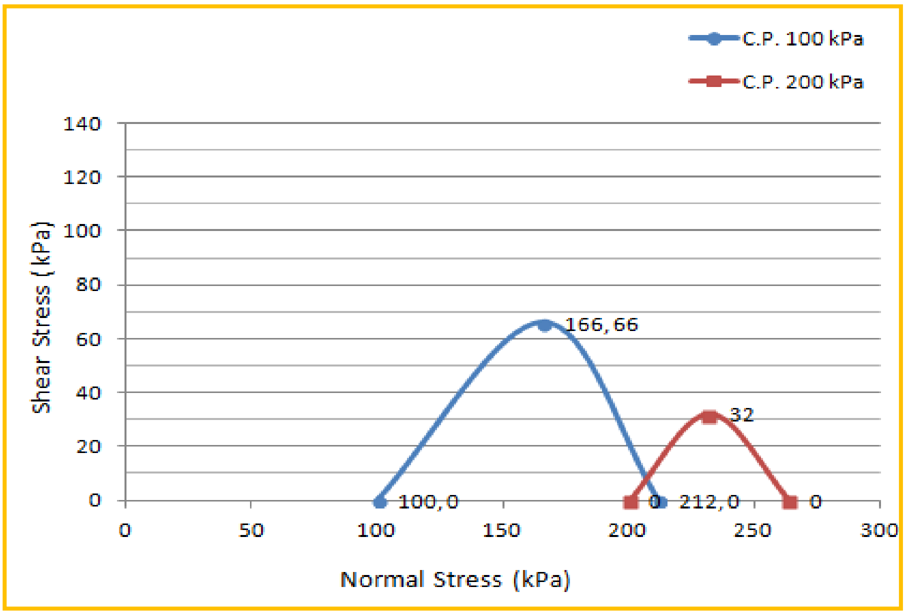


Figure 3.40 – Shear Stress Vs Normal Stress for mix

Unconsolidated Undrained Cohesion (C_u) = (66+32)/2 = 49 kPa

Unconsolidated Undrained Shear Strength (q_u) = 98 kPa

3.4.2 Summary of Laboratory Test Results

Hydraulic Conductivity and Compressive Strength are the predominant factors taking in to consideration when planning and designing Soil-Cement-Bentonite (SCB) like slurry base cutoff walls. So it is required to summarize those properties as illustrated in Table 3.8 and Table 3.9 respectively.

Table 3.8 – Summary of the Hydraulic Conductivity (k), values

MIX NO. & (B : C)%	**@ 7 day, k (m/sec) 10^{-10}**			**@ 14 day, k (m/sec) 10^{-10}**			**@ 28 day, k (m/sec) 10^{-10}**		
	Av.	Min.	Max.	Av.	Min.	Max.	Av.	Min.	Max.
1 *(3.3:8)*	8.1	1.8	20.9	5.5	0.5	23.7	1.9	0.8	4.9
2 *(2.2:4)*	5.7	3.7	10.8	4.9	1.7	10.7	1.7	0.7	3.4
A *(2.0:3)*	4.7	3.0	6.0	4.3	3.3	5.5	3.6	1.9	5.8
B *(1.0:3)*	3.0	1.9	4.6	2.5	1.3	3.9	1.5	0.6	3.1
Borrow Sample / Natural Soil							9.1	4.1	13.2

Table 3.9 – Summary of the Triaxial test results

MIX NO.	**UNCONSOLIDATED UNDRAINED COHESION (C_U) (kPa)**		**AVERAGE (C_U) (kPa)**	**COMPRESSIVE STRENGTH (q_u) (kPa)**
	@ Cell Pressure 100 kPa	*@ Cell Pressure 200 kPa*		
1	*78*	*105*	*91.5*	*183*
2	*26*	*55*	*40.5*	*81*
A	22.75	*29.5*	*26.125*	*52.25*
B	*66*	*32*	*49*	*98*

3.3.3 Numerical Modeling Results

SEEP/W Analysis

There is no enough borehole data to interpret a cross section of the critical section CH 470 m to CH 590 m. Hence models were defined assuming that profile of a single borehole prevails all over the dam cross section. Model 1 was defined assuming that profile of the borehole 1 is existing throughout the dam body. Likewise, model 2 & 3 were defined considering the borehole 2 & 3 respectively. The assumed sub surface profiles with respect to the borehole no1, 2 and 3 were analyzed for flux variation before and after the introduction of SCB cutoff wall.

Model 01

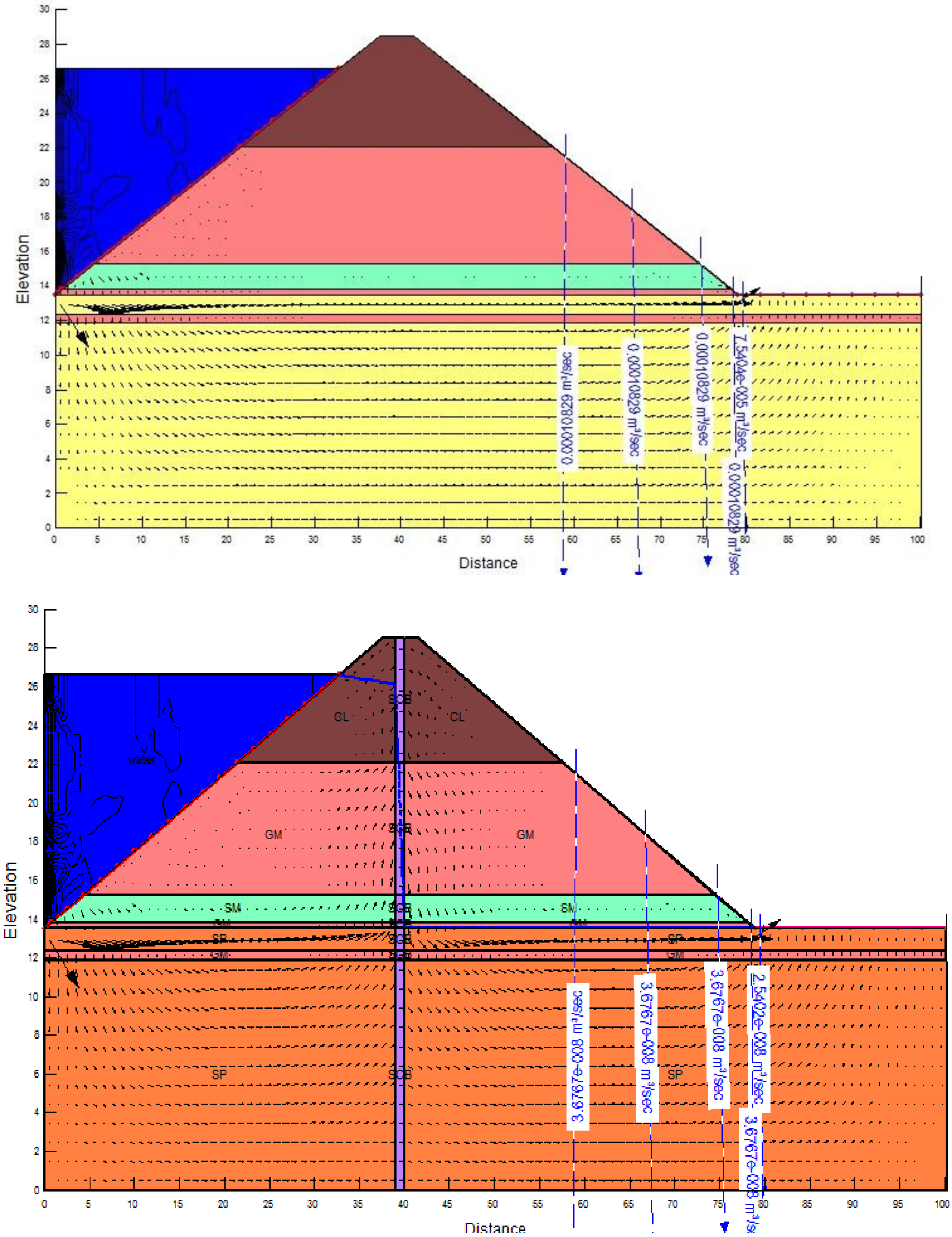
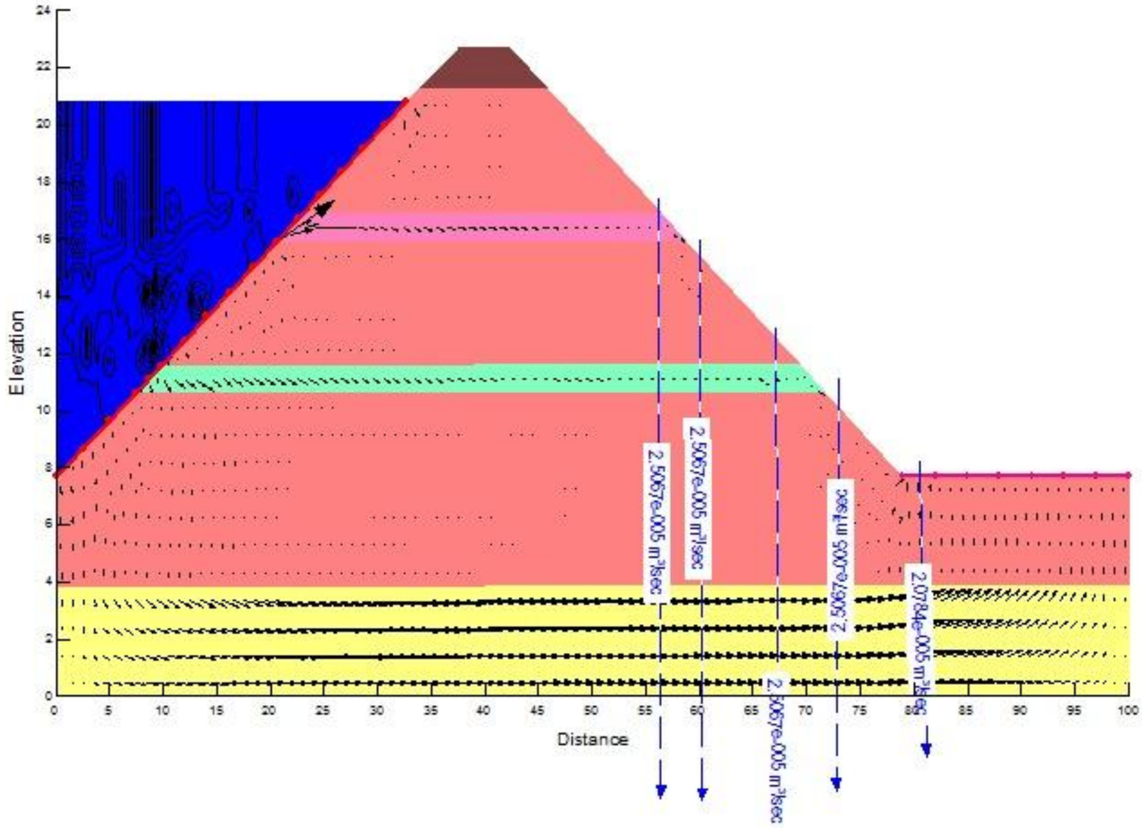


Figure 3.41 - Model 01 from BH 01 and seepage analysis with & without the SCB wall

Model 02



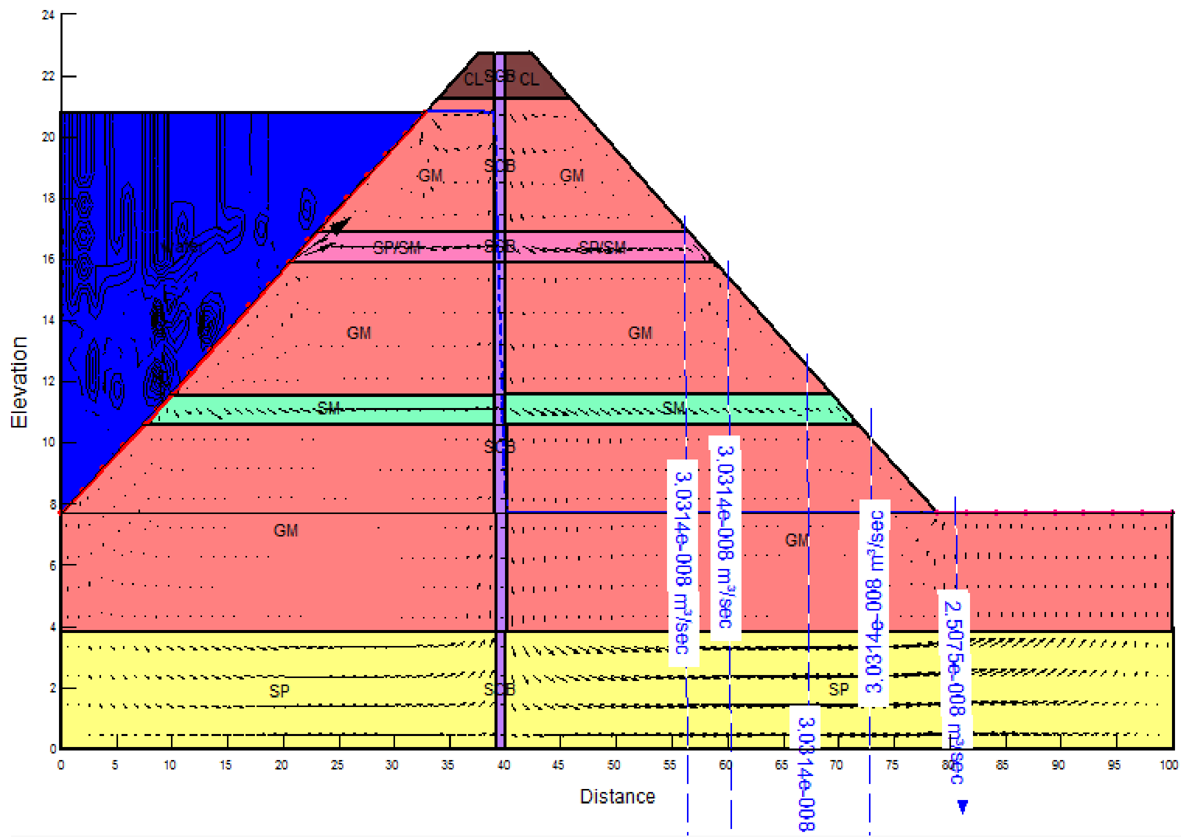


Figure 3.42 - Model 02 from BH 02 and seepage analysis with & without the SCB wall

Model 03

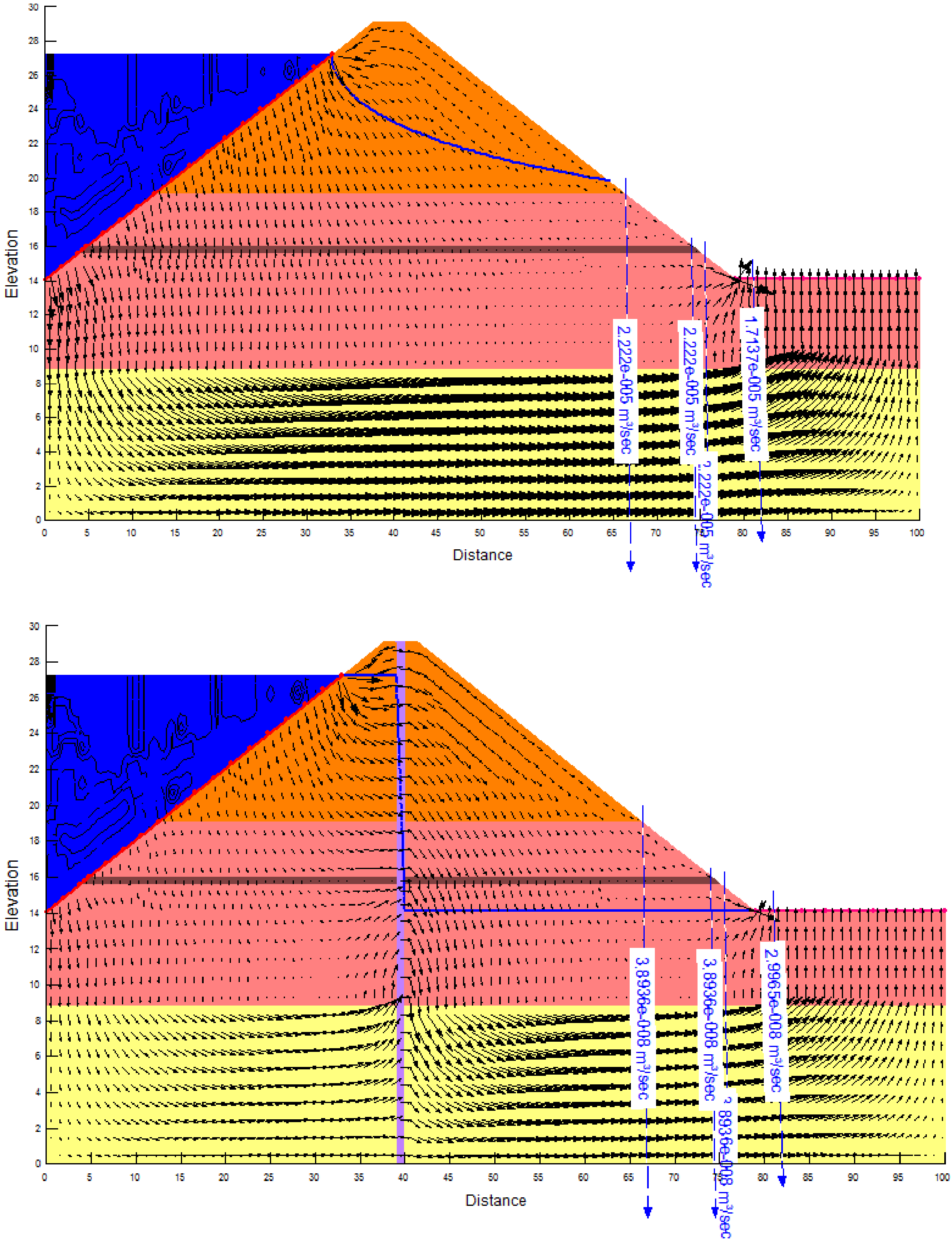


Figure 3.43 - Model 03 from BH 03 and seepage analysis with & without the SCB wall

Summary of SEEP/W Analysis

Model 01

Table 3.10 – Summary of Flux of model 01

Flux Section	Flux (m^3/sec)		Percentage of Flux Reduction (%)
	Without SCB cutoff wall	With SCB cutoff wall	
1	7.540×10^{-5}	2.5402×10^{-8}	99.966
2	1.0829×10^{-4}	3.6767×10^{-8}	99.9660
3	1.0829×10^{-4}	3.6767×10^{-8}	99.9660
4	1.0829×10^{-4}	3.6767×10^{-8}	99.9660

Model 02

Table 3.11 – Summary of Flux of model 02

Flux Section	Flux (m^3/sec)		Percentage of Flux Reduction (%)
	Without SCB cutoff wall	With SCB cutoff wall	
1	2.0784×10^{-5}	2.0507×10^{-8}	99.9013
2	2.5067×10^{-5}	3.031×10^{-8}	99.9879
3	2.5067×10^{-5}	3.031×10^{-8}	99.9879
4	2.5067×10^{-5}	3.031×10^{-8}	99.9879

Model 03

Table 3.12 – Summary of Flux of model 03

Flux Section	Flux (m^3/sec)		Percentage of Flux Reduction (%)
	Without SCB cutoff wall	With SCB cutoff wall	
1	1.7137×10^{-5}	2.9965×10^{-8}	99.8251
2	2.222×10^{-5}	3.8936×10^{-8}	99.9824
3	2.222×10^{-5}	3.8936×10^{-8}	99.9824
4	2.222×10^{-5}	3.8936×10^{-8}	99.9824

SLOPE/W Analysis

The assumed sub surface profiles with respect to the borehole no1, 2 and 3 were analyzed for static stability before and after the introduction of SCB cutoff wall.

Model 01

Factor of Safety Values of existing Vendrasan Dam

Table 3.13 – F.O.S. values *of existing Vendrasan Dam for Model 01*

Method of analyze	Factor of Safety	
	With respect to moment equilibrium (F_m)	With respect to force equilibrium (F_f)
Ordinary	1.755	-
Bishop	2.044	-
Janbu	-	1.866
Morgenstern-Price (M-P)	2.048	2.055

Factor of safety values after introduce the SCB cutoff wall

Table 3.14 – F.O.S. values after introduce the SCB cutoff wall for Model 01

Method of analyze	Factor of Safety	
	With respect to moment equilibrium (F_m)	With respect to force equilibrium (F_f)
Ordinary	2.140	-
Bishop	2.395	-
Janbu	-	2.193
Morgenstern-Price (M-P)	2.395	2.402

The critical slip surface of the above two cases are illustrated in the Figure 3.44 and 3.45 as modeled in the SLOPE/W.

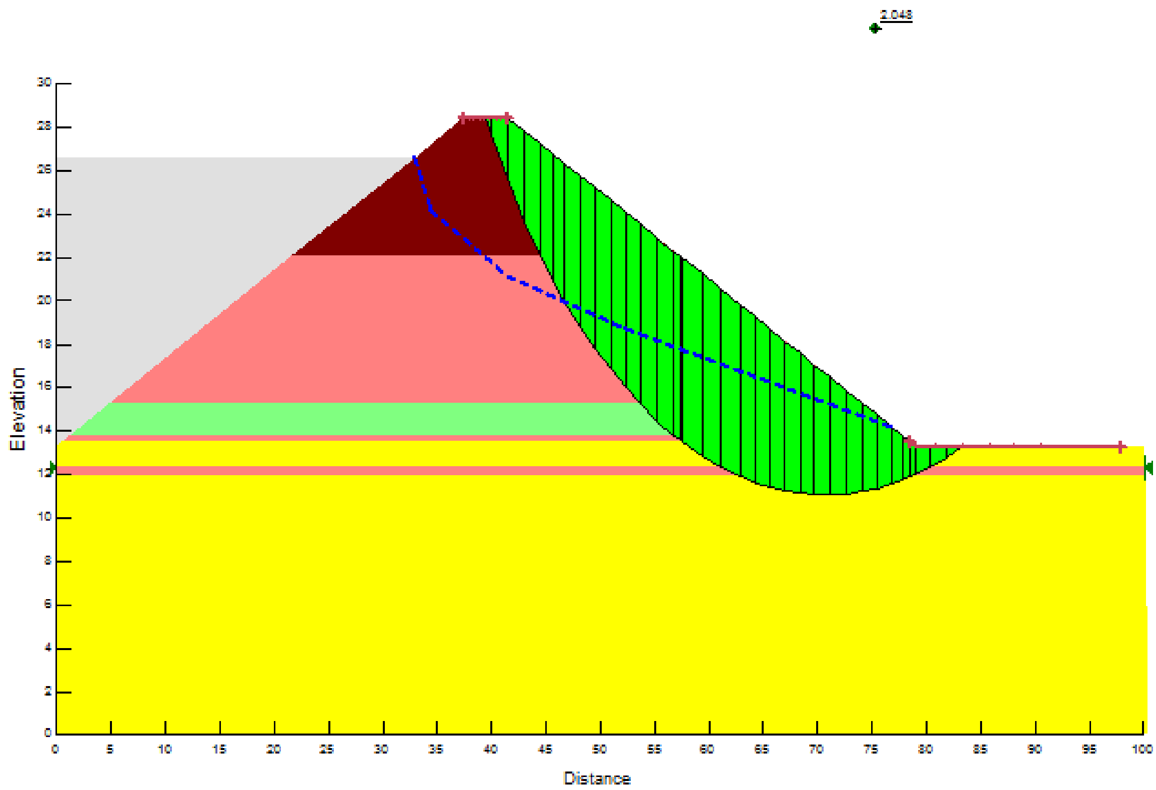


Figure 3.44 – Critical slip surface, without SCB wall

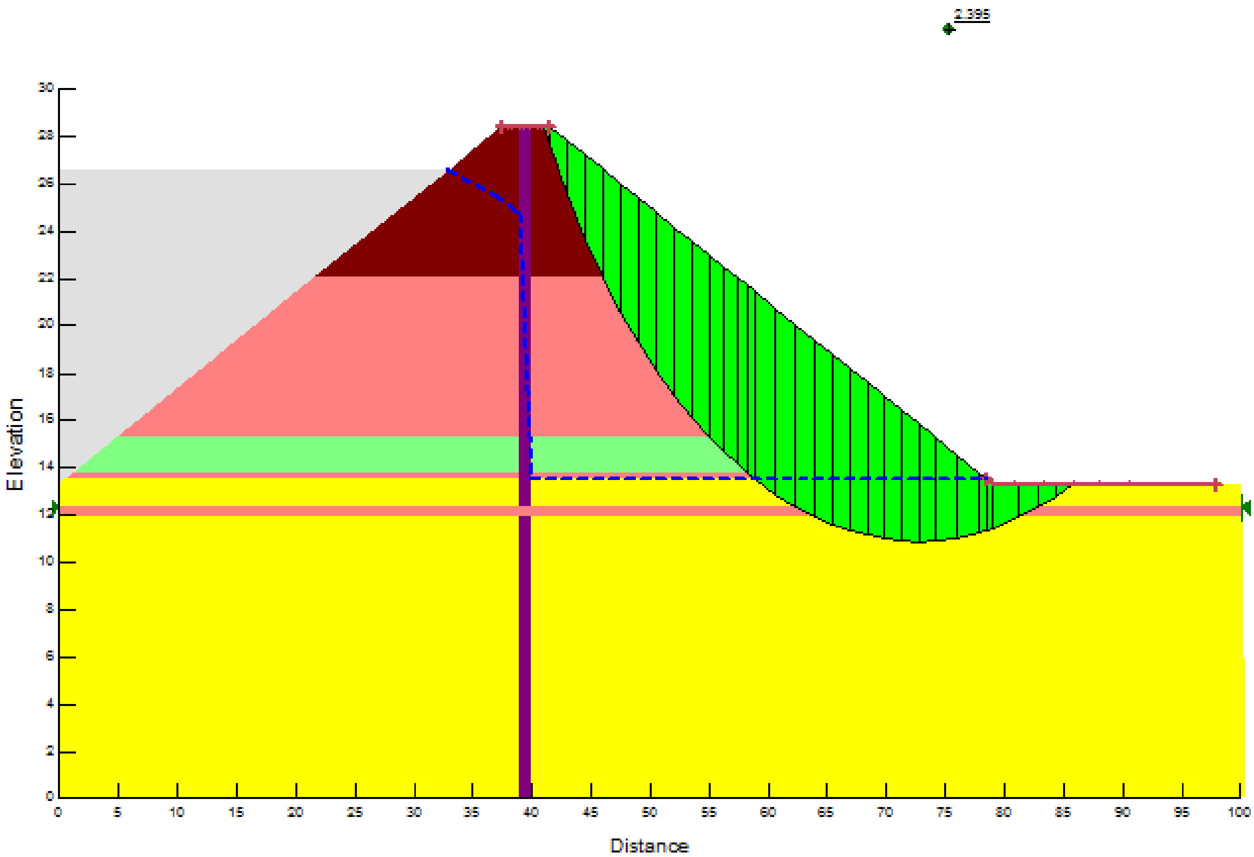


Figure 3.45 – Critical slip surface, with SCB wall

Model 02

Factor of Safety Values of existing Vendrasan Dam

Table 3.15 – F.O.S. values of existing Vendrasan Dam for Model

Method of analyze	Factor of Safety	
	With respect to moment equilibrium (F_m)	With respect to force equilibrium (F_f)
Ordinary	1.516	-
Bishop	1.688	-
Janbu	-	1.546
Morgenstern-Price (M-P)	1.687	1.692

Factor of safety values after introduce the SCB cutoff wall

Table 3.16 – F.O.S. values after introduce the SCB cutoff wall for Model 02

Method of analyze	Factor of Safety	
	With respect to moment equilibrium (F_m)	With respect to force equilibrium (F_f)
Ordinary	1.790	-
Bishop	1.943	-
Janbu	-	1.784
Morgenstern-Price (M-P)	1.939	1.945

The critical slip surface of the above two cases are illustrated in the Figure 3.46 and 3.47 as modeled in the SLOPE/W.

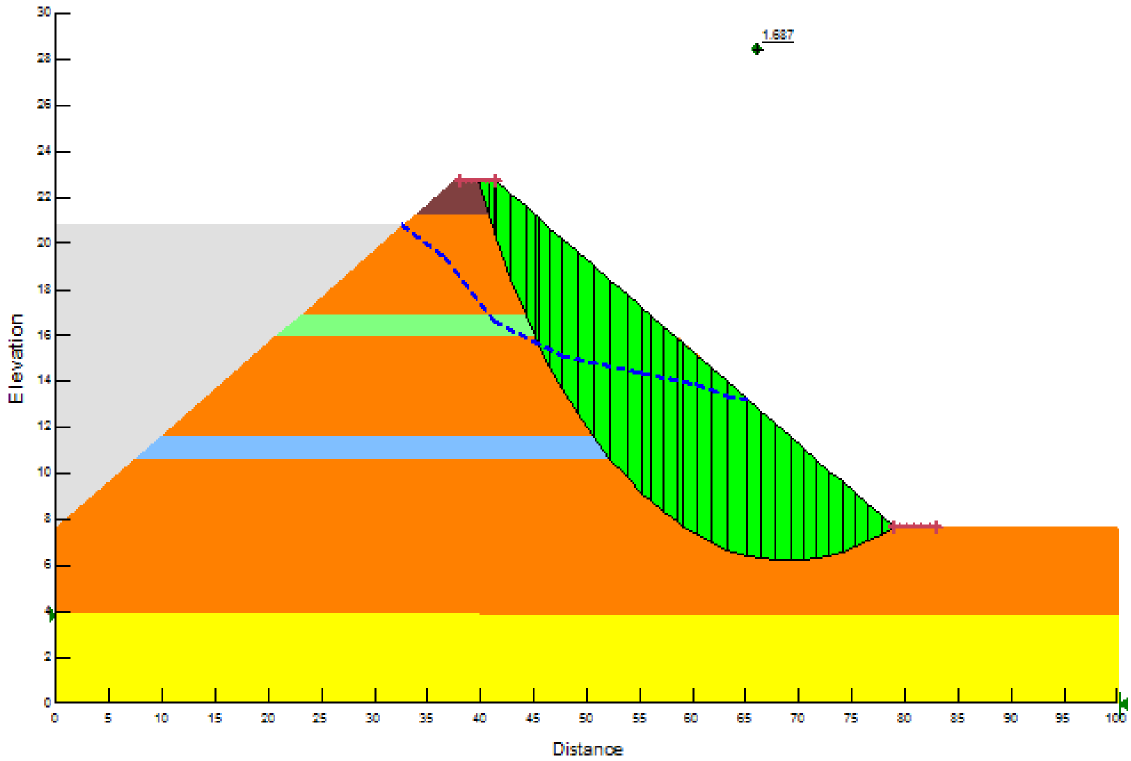


Figure 3.46 – Critical slip surface, without SCB

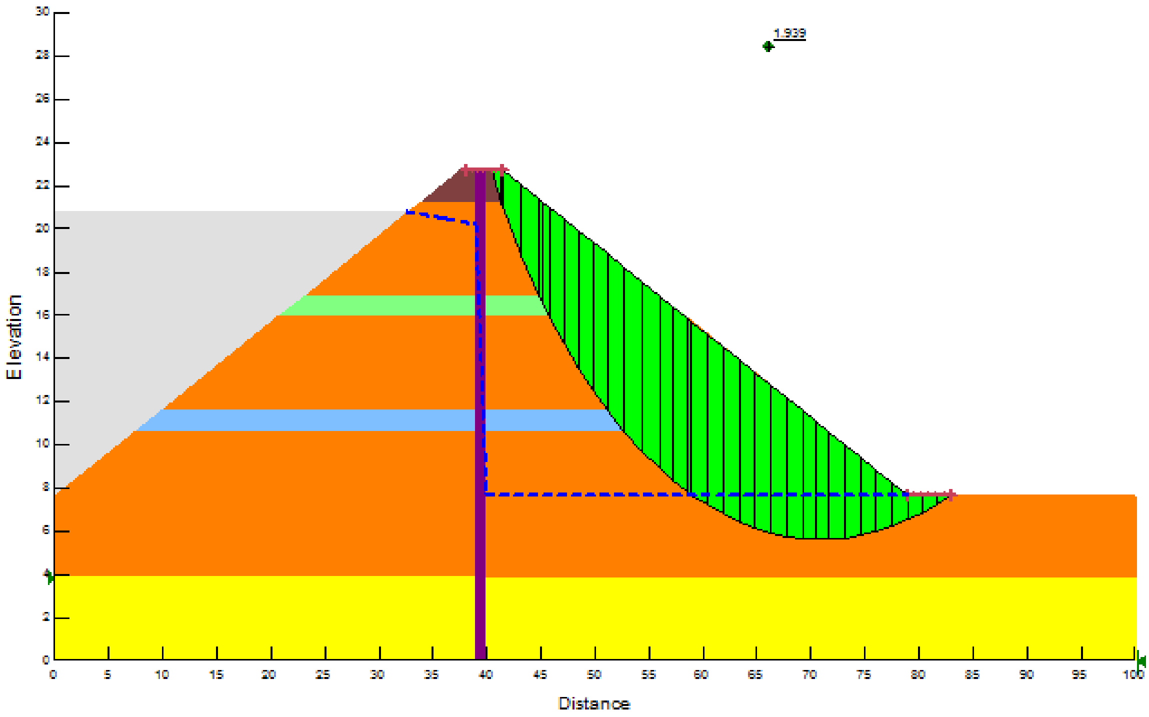


Figure 3.47 – Critical slip surface, with SCB

Model 03

Factor of Safety Values of existing Vendrasan Dam

Table 3.17 – F.O.S. values *of existing Vendrasan Dam for Model 03*

Method of analyze	Factor of Safety	
	With respect to moment equilibrium (F_m)	With respect to force equilibrium (F_f)
Ordinary	1.734	-
Bishop	1.846	-
Janbu	-	1.781
Morgenstern-Price (M-P)	1.851	1.852

Factor of safety values after introduce the SCB cutoff wall

Table 3.18 – F.O.S. values after introduce the SCB cutoff wall for Model 03

Method of analyze	Factor of Safety	
	With respect to moment equilibrium (F_m)	With respect to force equilibrium (F_f)
Ordinary	2.113	-
Bishop	2.167	-
Janbu	-	2.106
Morgenstern-Price (M-P)	2.172	2.173

Critical slip surface of the above two cases are illustrated in the Figure 3.48 and 3.49 as modeled in the SLOPE/W.

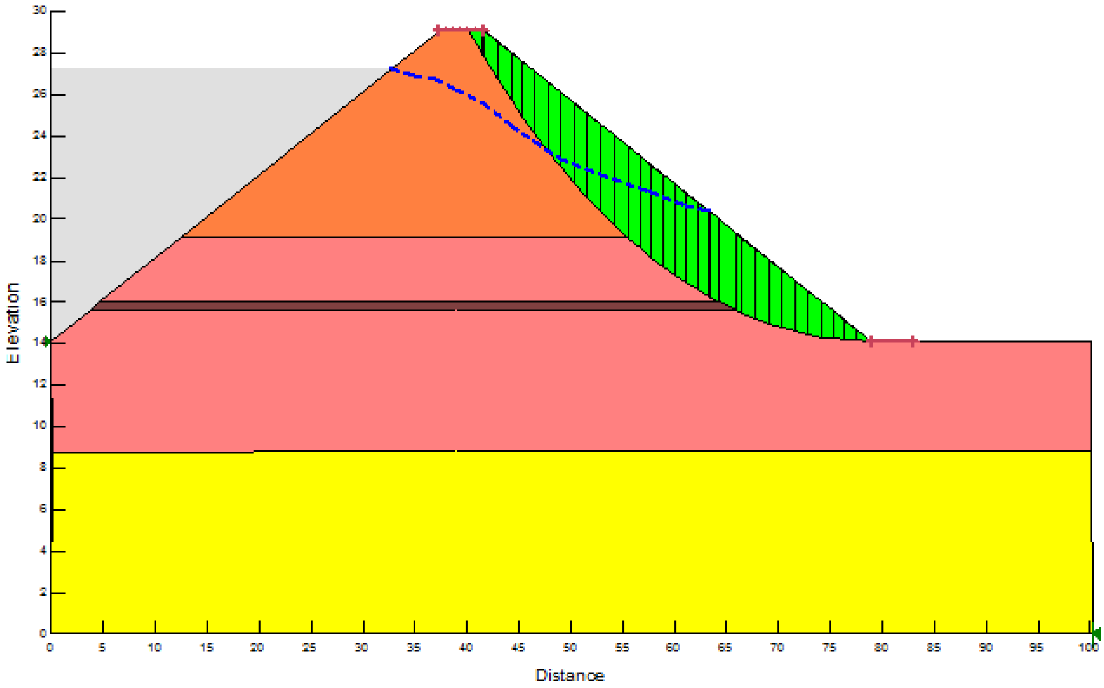


Figure 3.48 – Critical slip surface, without SCB wall

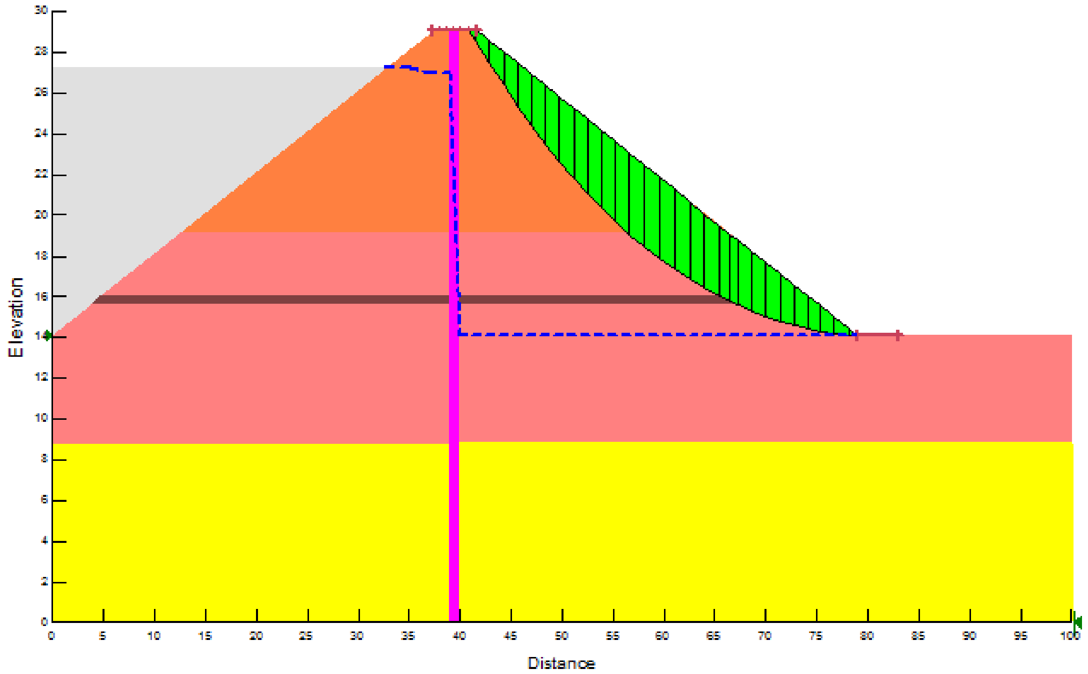


Figure 3.49 – Critical slip surface, with SCB wall

3.5 Discussion and Recommendation

The following observations could be made based on the detail study on the results of the series of laboratory testing.

Coefficient of Consolidation (C_v), Coefficient of Volume Compressibility (m_v) and Hydraulic Conductivity (k) values comparatively reduce with time for all the four (4) mix proportions. When consider the Soil-Cement-Bentonite (SCB) slurry cutoff wall consistency, hydraulic conductivity (k) is the utmost important property. The variation of the k, clearly illustrate with the summary Table 3.8.

Comparing with the borrow material of composite 2, all the mix proportions show considerable impermeability improvement with time.

The plotted graphs disclose hydraulic conductivity values generally decrease with the loading increments. It provides evidence that permeability values are decreasing with increasing confining stresses.

When compare the mix no.1 and 2 from same composite (composite 1) mix no.2 exhibits lower permeability while mix no.1 exhibits higher strength values. This may due to the higher cement content action on bentonite. Portland cement chemically affects the ability of bentonite to “swell” and help retain water, creating more porous backfill material.

When compare the mix no. A and B from same composite (composite 2) and same cement percentage, mix no. B exhibits lower permeability and higher strength values which has lower bentonite percentage than mix no. A. Bentonite seems to be proportionately influencing on permeability. This behavior is not fit with the “relationship between permeability and quantity of bentonite added to Soil-Bentonite (SB) backfill” presented by D’Appolonia, illustrated in Figure 2.4. But fairly fit with the results in case study 2 presented in section 2.2.3 (Mix no. 3B, 5B, 6B).

When compare the mix no. 2 and mix no. B from composite 1 and composite 2, mix no 2 shows relatively low permeability values which is from composite 1 and has higher fine content. Fine content seems to be inversely influencing on permeability. It also fairly fit with the “permeability of soil-bentonite backfill related to fines content” presented by D’Appolonia, as illustrated in Figure 2.5.

According to the United State Army Corps of Engineers (USACOE) specifications on soil-cement-bentonite, permeability criteria shall be less than 5×10^{-7} cm/sec and compressive strength criteria shall be in the range of 100-700 kPa.

Detail study on the numbers in Table 3.8 concerning all minimum, maximum and average hydraulic conductivity values, unveil the best material proportion is mix No. B and values are gradually improving with time. Further it satisfies the USACOE specifications.

So, it is clear that Soil-Cement-Bentonite (SCB) backfill material is suitable to introduce to impede seepages. Mix no.B slurry cutoff backfill material can be recommended to utilize in Vendrasan Dam, Trincomalee.

Further, numerical modeling of the dam with three assumed models, reveal the very important facts of seepage quantities and static stability. The summary Tables 3.10, 3.11 and 3.12 clearly show the flux reduction through all the selected sections after the implementation of SCB cutoff wall. Percentage flux reduction is very close to the 99% in numerically.

The summary Tables 3.13 to 3.18 disclose the improvement of factor of safety values which were already satisfy the required minimum factor of safety value (1.5) at static and steady state condition of downstream of the dam.

So, it is clear the suitability and the applicability of Soil-Cement-Bentonite Slurry Cutoff Wall material and its performance with respect to seepage and stability.

3.5.1 Recommendations to further studies

This study area is very gigantic and complex. So studies can be focus in to various scenarios and few are stated below.

- SCB walls are new to Sri Lankan engineering context, so study can be extended to study on other SCB applications like excavation support, salt water intrusion, flood control and waste water management etc.
- Various other materials like recycled tire shreds can also be introduced to the backfill in order to improve backfill properties. Research can be done on searching additives to introduce to SCB backfill material to improve its engineering properties.
- SCB slurry walls can be structurally supported for upgrading and modifying as a stronger structural wall.

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